MAGNITUDE AND FREQUENCY OF FLOODS FROM URBAN STREAMS IN LEON COUNTY, FLORIDA

By Marvin A. Franklin and Gerald T. Losey

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ABBREVIATIONS AND CONVERSION FACTORS

The inch-pound units used in this report may be converted to metric units (SI) by the following conversion factors:

Multiply inch-pound unit	By	To obtain metric unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
foot per mile (ft/mi)	0.1894	meter per kilometer (m/km)

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ABSTRACT

Techniques are provided for estimating flood magnitudes for urban-flow streams in Leon County, Florida, for recurrence intervals of 2, 5, 10, 25, 50, 100, and 500 years. Synthetic flood peaks were generated by using a calibrated lumped-parameter rainfall-runoff model, pan evaporation data from Milton, Florida, and long-term unit rainfall records from Thomasville-Coolidge, Georgia, and Pensacola, Florida. The synthetic flood peaks were used to develop station flood-frequency relations which were used in multiple linear regression analyses to derive regional equations relating flood magnitude to basin characteristics. Significant basin characteristics were drainage area, impervious area, and geographic location. The average standard error of prediction ranged from ±32 percent for the 5-year recurrence interval to ±47 percent for the 500-year recurrence interval flood.

INTRODUCTION

A knowledge of flood characteristics is essential for designing drainage structures and for using flood-prone land. A reliable estimate of flood magnitude and frequency is necessary to design economical structures and prepare realistic zoning ordinances for a community. Leon County and the city of Tallahassee have a history of local flooding resulting from intense and generally brief storm events. This is evidenced by the intense rainfall of October 6, 1976, when 6 inches, mostly within 1 hour, fell on the city. Floods cause property damage and on occasion result in loss of life, as occurred on May 17, 1974, when two teenage boys drowned. The most recent flooding occurred March 6, 1983, when an intense storm moved across the county producing a total of 5.22 inches of rain.

Recognizing the need for reliable flood data and improved techniques for estimating the frequency and magnitude of flooding, the U.S. Geological Survey and the Public Works Department of Leon County, Fla., began a cooperative investigation in 1978 that included installation of a network of streamflow and rainfall gages and collection and analysis of flood data in Leon County. As a result, 15 continuous-precipitation gages, 2 continuous-record gaging stations, and 14 partial record gaging stations were installed in 1979 to collect storm data. Figure 1 shows the location of the rainfall and stage-discharge collection sites. Table 1 gives gage locations, map location numbers, and station numbers. The station number is the identification under which the data are stored in the WATSTORE (National Water Data Storage and Retrieval System) unit and daily values files.

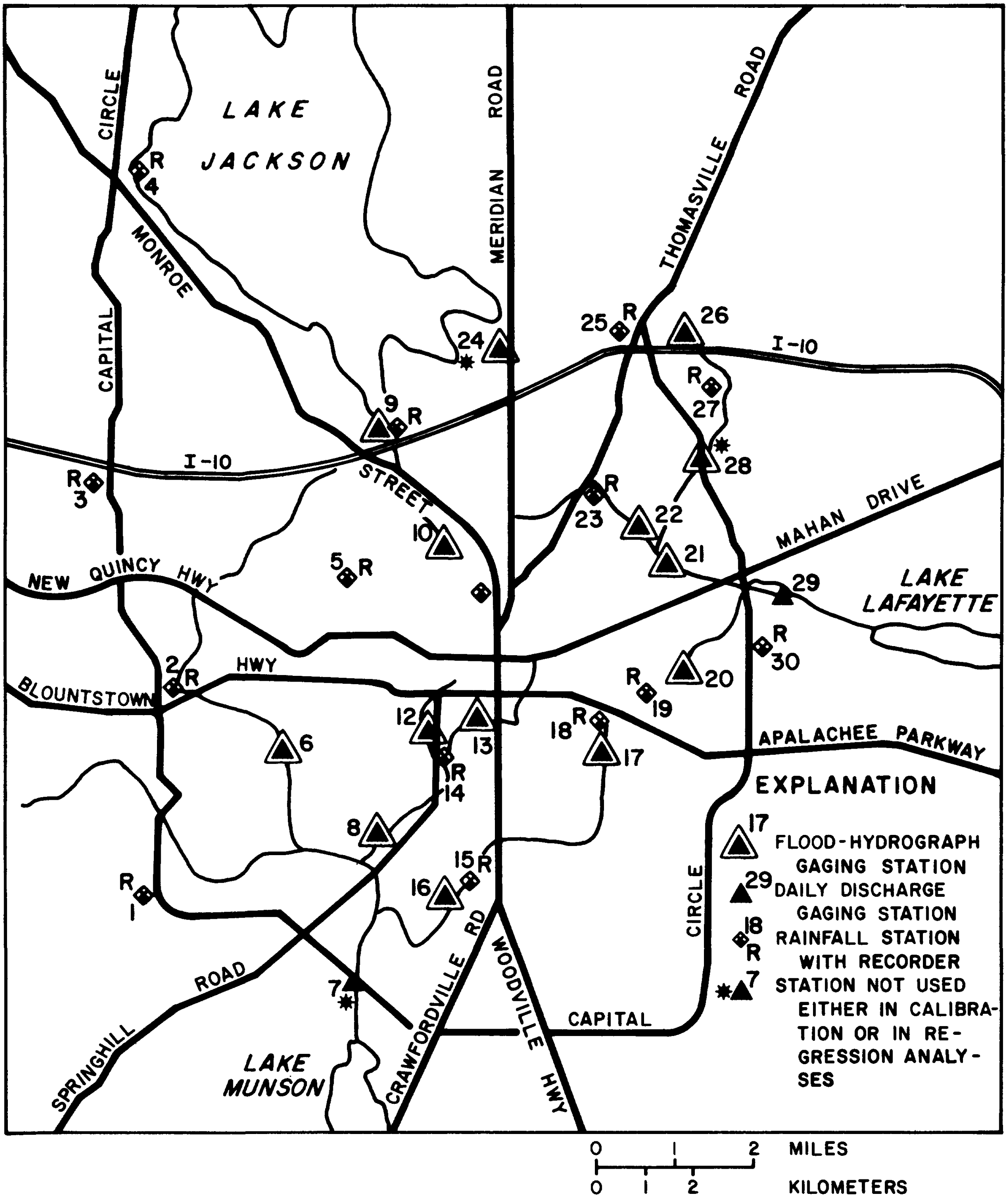

Figure 1.--Location of rainfall- and discharge-collection sites in the Tallahassee area of Leon County, Florida.

Table 1.--Gage identification and location

Map location No.	Station identification No.	Type and location
1	302347084212300	Rainfall gage at Tallahassee Municipal Airport near National Weather Service rain gage.
2	302609084211000	Rainfall gage behind Wayne Coloney plant near intersection of Blountstown Highway and Capital Circle.
3	302842084215200	Rainfall gage just west of Capital Circle near intersection with Commonwealth Boulevard.
4	303200084212500	Rainfall gage in front of Sunset Fish Camp near end of Lake Drive.
5	302731084191600	Rainfall gage near east end of lake between San Luis Road and Ocala Road.
6	02327012	Discharge gage on left bank upstream from bridge on Roberts Avenue over west side drainage ditch near intersection with Mabry Street.
7	02327017	Discharge gage on downstream side of bridge on Capital Circle over Munson Slough.
8	02327015	Discharge gage on right upstream end of culvert over central drainage ditch on Orange Avenue near Springhill Road.
9	02329186	Discharge and rainfall gage on right downstream end of culvert over Megginnis Arm Tributary on Meginnis Arm Road near Interstate-10.
10	02329181	Discharge gage on right bank 20 feet upstream from detention culvert behind Northwood Mall and adjacent to Boone Boulevard.
11	302731084165400	Rainfall gage in north parking lot of the old National Guard Armory between Seventh and Eighth Avenues.

Table 1.--Gage identification and location--Continued

Map location No.	Station identification No.	Type and location
12	02327013	Discharge gage on left bank downstream of bridge over central drainage ditch on Airport Drive at intersection with Eppes Drive.
13	02327014	Discharge gage on left bank upstream of bridge over St. Augustine Branch on Wahnish Way at intersection with Canal Street.
14	302536084180500	Rainfall gage attached to north wall of sewage disposal plant at intersection of Gamble Street and Lake Bradford Road.
15	302438084172400	Rainfall gage under electrical transmission lines adjacent to Wahnish Way and east drainage ditch.
16	02327016	Discharge gage on downstream side of bridge over east drainage ditch on Bragg Drive.
17	302549084152900	Discharge gage on left bank upstream of culvert over east drainage ditch on Apakin Nene in Indian Head Acres.
18	302601084153600	Rainfall gage near electrical substation between Ostin Nene and Chowkeebin Nene in Indian Head Acres.
19	302622084145900	Rainfall gage on dam of Governor's Square detention pond adjacent to Blairstone Road.
20	02326842	Discharge gage on left bank upstream of culvert over Governor's Square drainage ditch on Park Avenue near intersection with Blairstone Road.
21	02326838	Discharge gage on right bank upstream of culvert over northeast drainage ditch on Miccosukee Road near intersection with Doomar Drive.

Table 1.--Gage identification and location--Continued

Map location No.	Station identification No.	Type and location
22	02326836	Discharge gage on right bank upstream of culvert over McCord Park Pond drainage ditch on Centerville Road near intersection with Trescott Drive.
23	302822084154400	Rainfall gage near west side of pond in McCord Park between Trescott Drive and Armistead Road.
24	02329161	Discharge gage on left bank of Fords Arm Tributary downstream of Meridian Road near intersection with Lexington Road.
25	303010084151200	Rainfall gage inside fence enclosure south side of Timberlane Shops on the Square adjacent to Interstate-10.
26	02326825	Discharge gage on left bank upstream of culvert over northeast drainage ditch on Hadley Road near intersection with Raymond Diehl Road.
27	302935084142100	Rainfall gage at southeast end of Wembley Way in Eastgate.
28	02326828	Discharge gage on right upstream end of culvert over northeast drainage ditch on Capital Circle at intersection with Centerville Road.
29	02326845	Discharge gage near upstream right end of wier across northeast drainage ditch just upstream of Weems Road.
30	302707084132400	Rainfall gage inside enclosure of National Guard Armory near Federal Correctional Institution.

The objectives of this investigation were to collect hydrologic data from selected drainage systems in the urban areas of the county, to analyze the data, and to derive regression equations that can be used to estimate the magnitude and frequency of floods in urban parts of the county. The purpose of this report is to present:

1. Methods used in the collection of the data for the study;

2. Methods used in the analysis of the data and the results of that analysis;

3. Regression equations needed to estimate the magnitude of a selected recurrence-interval flood;

4. A step-by-step example to illustrate use of the information.

More than 25 years of observed peak-flow data are generally needed for reliable estimates of the 50- and 100-year floods at a stream-gaging site. To reduce the time required for data collection, runoff and rainfall data collected in this investigation were used (Franklin, 1982) to calibrate a lumped-parameter, rainfall-runoff model. Results of the model were extended to generate long-term flood records from long-term rainfall data furnished by the National Weather Service.

Log-Pearson type III frequency analysis performed on this synthetic data base generated flood-frequency data for each gaging station. The majority of locations where flood-frequency information is needed are ungaged; therefore, multiple linear regression analyses were performed to derive regional relations between flood discharge and selected basin characteristics.

The equations presented in this report are applicable in the urban parts of Leon County. Bridges (1982, p. 9-11) presented equations for rural areas of the State. Sauer and others (1981) presented methods of using rural equations and an urban development factor to compute flood discharge for urban areas where little data are available.

This report was prepared by the U.S. Geological Survey under the cooperative program with the Leon County Public Works Department, Russell J. Tagliarini, Administrator. Valuable assistance was provided by the Department in location and construction of the data-collection network.

V. B. Sauer, U.S. Geological Survey, Atlanta, Ga., provided valuable technical advice and assistance. The dedicated effort of coworkers from the Survey's Tallahassee Subdistrict Data Section in the collection and processing of the data is gratefully acknowledged.

DATA ACQUISITION

Data acquisition was divided into two phases. The first phase required establishment of gaging stations for collection of storm-rainfall and flood-runoff data on streams in the study area.

The second phase required collection or measurement of independent basin characteristics for use in multiple-regression analyses to define common parameters in each basin that could be related to flood magnitudes. Approximately 75 percent of the effort of this investigation was directed toward the acquisition and processing of data.

Rainfall and Runoff

The rain gage system consists of a collector that is the same in surface area as the National Weather Service standard 8-inch diameter rain gage. A typical rainfall-collection site is shown in figure 2. Rain falling into the collector drains to a vertical 3-inch diameter galvanized pipe. The storage volume of the pipe is sufficient to hold a 17-inch rainfall. A float in the pipe is connected to a digital recorder that punches a 16-channel paper tape with the equivalent rainfall depth at 5-minute intervals. By using a calibrated float wheel, the cumulative rainfall is recorded in inches. Unit rainfall in a given 5-minute interval is computed by subtracting the quantity recorded at the start of the 5-minute interval from the quantity recorded at the end of that interval. The daily rainfall total is computed by subtracting the first reading of the day from the last reading of the day. Each 5-minute value of rainfall is stored in the computer for storm-event days. On days when rainfall occurs, but no runoff is produced, only the daily total is stored.

Figure 2.--Typical rainfall data-collection site.

The stream stage recorder also works on the float and digital-recorder system. A typical discharge data collection site is shown in figure 3. A 10-inch stilling well is set in the streambank and connected to the stream with intakes. Stage (or elevation of the water surface) is recorded at 5-minute intervals. Each stage and rainfall site is visited every 2 to 3 weeks to insure proper operation and to remove the 16-channel punch tape for processing the record. From the 16-channel punch record of stage and the stage-discharge relation, discharge for each 5-minute interval is computed and stored in the computer.

Figure 3.--Typical discharge data-collection site.

Whenever possible, discharge measurements are made during runoff events to develop stage-discharge relations at each site. This involves making measurements of depth and velocity at several sections across the stream. The area of flow is computed from the depth observations and the measured width of each section. The area is then multiplied by the velocity to give the discharge in cubic feet per second; discharges for all sections are added to determine total discharge for the measurement; and this discharge is plotted against the mean stage of the stream at the time of measurement. A stage-discharge relation for the central drainage ditch at Airport Drive is illustrated in figure 4.

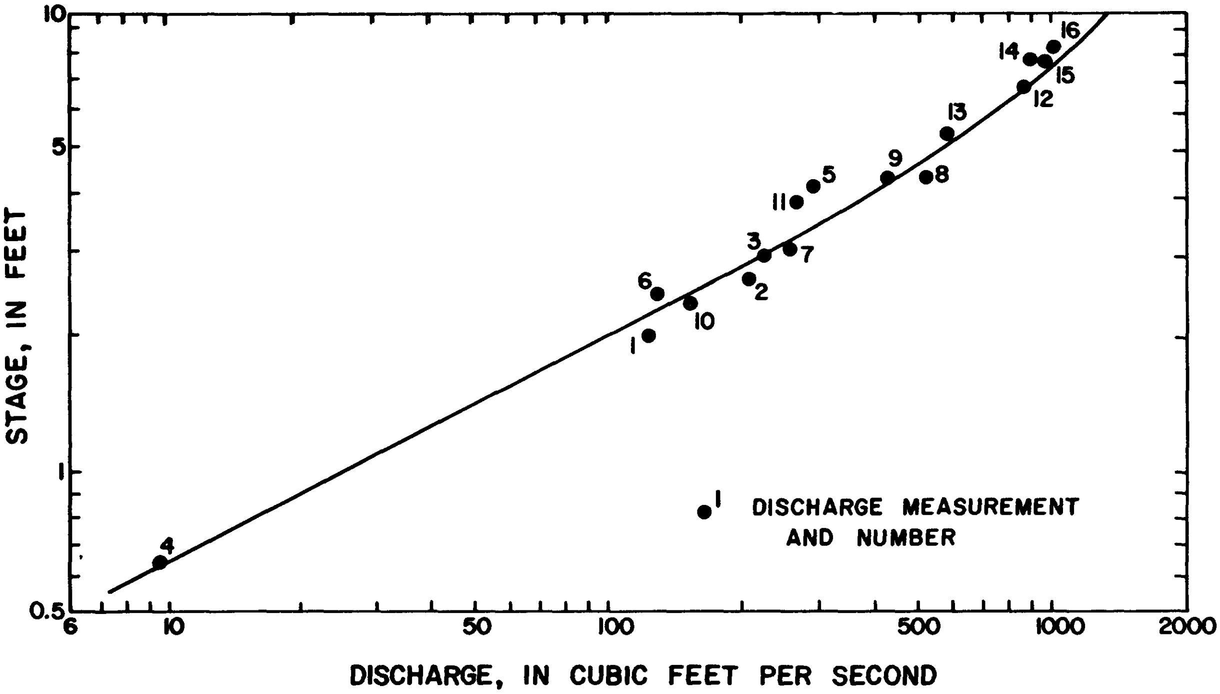

Figure 4.--Stage-discharge relation, central drainage ditch at Airport Drive (map number 12).

Basin Characteristics

The basin characteristics tested for significance in the multiple regression analyses are defined below. The observed ranges in values are given in parenthesis, and data for each basin are given in table 2.

Drainage area, (DA), in square miles (0.21 to 15.9): the contributing drainage area was planimetered from U.S. Geological Survey 7½-minute topographic maps. Adjustments were made for areas that crossed natural divides as a result of storm sewers or streets.

Main-channel length, (L), in miles (0.58 to 6.50): the length between the gage and the basin divide.

Main-channel slope, (SL), in feet per mile (11.9 to 128): the average slope between points 10 percent and 85 percent of the main-channel length measured from the gage to the basin boundary. The altitude of the points were taken from the best available topographic map.

Storage, (ST), in percent (0.0 to 4.26): the area of lakes, ponds, and swamps in the contributing drainage area.

Impervious area, (IA), in percent (5.8 to 54): the area of impervious surface in the basin. The impervious area for each basin was determined by subdividing the basin into land-use types. The percentage of impervious area for each land-use type was field checked. The area for each type was determined by planimetering. The value was checked by using the grid method.

<u>Basin development factor</u>, (BDF), (0 to 8): the sum of all street and channel index numbers (Sauer and others, 1981). Values of BDF can vary from zero to 12. A value of zero does not necessarily mean the watershed is completely nonurban because watersheds that have all index numbers of zero may have housing, streets, and other developmental features that create impervious areas. The BDF was highly significant in a previous study of urban flood-peak discharge (Lopez and Woodham, 1983, p. 16) in the Tampa area.

Table 2.--<u>Basin characteristics</u>

Map location No.	Drainage area, DA, in square miles	Impervious area, IA, in percent	Main-channel length, L, in miles	Main-channel slope, SL, in foot per mile	Storage, ST, in percent	Basin development factor, BDF
6	15.9	8.8	6.48	11.9	4.26	2
8	8.11	27.0	4.69	18.1	1.13	6
9	3.44	28.3	2.94	32.0	0.11	6
10	0.26	43.0	0.90	65.6	0.00	7
12	3.29	48.5	2.34	45.3	0.35	8
13	2.06	54.0	2.72	32.2	0.05	8
16	5.40	19.6	4.41	18.6	1.70	4
17	0.21	25.0	0.58	128	0.00	7
20	1.04	25.0	1.28	82.0	3.01	4
21	9.83	23.3	5.26	16.2	2.71	3
22	2.92	31.2	2.50	32.8	1.88	5
24	1.66	5.8	1.84	46.2	1.08	0
26	0.79	20.3	1.23	54.5	2.54	0
28	3.85	23.0	3.63	26.2	1.19	0
29	15.7	22	6.50	12.9	2.77	2

The following description of how to determine BDF is based on information in a report by Sauer and others (1981).

Lines subdividing the basin into thirds were drawn so that the "upper third" of the basin included approximately one-third of the contributing drainage area that drained the upper reaches of the basin. Similarly, the "middle third" of the basin contained approximately one-third of the contributing drainage area that drained the middle reaches of the basin. The remaining lower one-third of the contributing drainage area is the "lower third" and drains the lower reaches of the basin. Travel time of flow was given consideration in drawing lines separating basin thirds. Therefore, distances along main streams and tributaries were marked to help locate the boundaries of the basin thirds so that within each third the travel distances of two or more streams are about equal. Precise definition of the lines subdividing the basin into thirds was unnecessary for the variables which utilize this concept. Therefore, the lines can generally be drawn on the drainage map by visual estimate without the need for measurements. Complex basin shapes and drainage patterns require more judgment when subdividing.

Within each subarea, four conditions of the drainage system are evaluated and assigned a code according to the following descriptions:

1. Channel improvements.--If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channel and principal tributaries (those that drain directly into the main channel), then a code of one (1) is assigned. Any one, or all, of these improvements would qualify for a code of one (1). To be considered prevalent, at least 50 percent of the main drainage channel and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, than a code of zero (0) is assigned.

2. Channel linings.--If more than 50 percent of the main drainage channel and principal tributaries have been lined with an impervious material, such as concrete, then a code of one (1) is assigned to this condition. If less than 50 percent of these channels are lined, then a code of zero (0) is assigned. The presence of channel linings would obviously indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.

3. Storm drains or storm sewers.--Storm drains are enclosed drainage structures (usually pipes) frequently used on the secondary tributaries where the drainage is received directly from the streets or parking lots. Quite often these drains empty into the main tributaries and channels which are either open channels, or in some basins, are also enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a subarea (third) consists of storm drains, then a code of one (1) is assigned to this condition, and conversely if less than 50 percent of the secondary tributaries consists of storm drains, then a code of zero (0) is assigned. If 50 percent or more of the main drainage channels and principal tributaries are enclosed, then the conditions of channel improvements and channel linings would also be assigned a code of one (1).

4. Curb and gutter streets.--If more than 50 percent of a subarea (third) is urbanized (covered by residential, commercial, or industrial development), and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, then a code of one (1) should be assigned to this condition. Otherwise, assign a code of zero (0). Frequently, drainage from curb and gutter streets will empty into storm drains.

The above guidelines for assigning the various drainage system codes are not intended to be precise measurements. A certain amount of subjectivity will necessarily be involved, however, field checking should be performed to obtain the best estimate. The basin development factor (BDF) is computed as the sum of the assigned codes. Obviously, with three subareas (thirds) per basin, and four drainage conditions to which codes are assigned in each subarea, the maximum value for a fully developed drainage system would be 12. Conversely, if the drainage system has not been developed, then a BDF of zero (0) would result. Such a condition does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, have some improvement of secondary tributaries, and still have an assigned BDF of zero (0).

ANALYTICAL TECHNIQUES

The analysis of flood data was divided into two phases. During the first phase, frequency distributions were determined from synthesized discharge records at gaged sites to estimate magnitude and frequency of flooding. During the second phase, multiple-regression analyses were made to extend the synthesized flood data to ungaged sites. The flood data were systematically related to the most significant factors that influence flood discharge. Factors for the final equations were selected based on the smallest regression error and practical application of the equations.

Long periods of gaged records are needed to make reliable estimates of the larger recurrence-interval floods (50- and 100-year). For all stations used in this report, the length of observed record was too short to produce reliable flood-frequency estimates. To improve that reliability, a U.S. Geological Survey rainfall-runoff model was used to extend the observed data collected during this investigation into a synthesized long-term record.

Rainfall-Runoff Model

The rainfall-runoff model developed by Dawdy and others (1972), with modifications described by Carrigan and others (1977), was used in this investigation. It combines soil-moisture-accounting and rainfall-excess components with the Clark (1945) flood-routing method. This lumped parameter model has three basic components: antecedent moisture, infiltration, and rainfall excess and routing. The Thiessen method was used to distribute rainfall over those basins with more than one rain gage. Excess rainfall was routed to the outlet of the basin from 20 time-of-travel bands for which percent impervious area was determined.

The antecedent soil-moisture component assesses the change in soil moisture based on daily rainfall and evaporation. Four parameters are used to simulate continuous antecedent soil moisture. Dawdy and others (1972) describe these parameters as follows:

1. EVC, a pan coefficient for converting measured pan evaporation to potential evapotranspiration;

2. RR, the proportion of daily rainfall that infiltrates into the soil;

3. BMSM, a maximum effective amount of base-moisture storage at field capacity, in inches; and

4. DRN, a coefficient controlling the rate of drainage of the infiltrated soil moisture, in inches per day.

The output from this component is the amount of base-moisture and infiltrated-surface-moisture storage.

The infiltration component uses the input of storm rainfall and output from the soil-moisture accounting component that indicated soil moisture at the beginning of the storm rainfall. Three parameters are used in the modified Philip (1954) infiltration equation to compute infiltration in the basin.

1. PSP, the suction at the wetted front for soil moisture at field capacity, in inches of pressure;

2. RGF, the ratio of the suction of the wetted front for soil moisture at wilting point to that of field capacity; and

3. KSAT, the effective saturated value of hydraulic conductivity used to determine infiltration rates, in inches per hour.

Rainfall excess computed in the infiltration component is routed to the outlet of the basin. The model uses a modification of the Clark flood-routing as described by Carrigan (1973). Three parameters are used in this step.

1. T_p, time to peak, in minutes;

2. T_c, basin lag time; and

3. KSW, a time characteristic for linear reservoir routing.

Model Calibration

Generally, about 40 significant storm events are needed at a rainfall-runoff site to achieve an optimum calibration of the model. However, a successful calibration can be achieved with less. For the period April 1979 to September 1982, a total of 323 events were recorded at the 15 streamflow sites used in model calibration. Many of these events, however, were not used in model calibration. Two reasons for not using flood events were: (1) peak discharge recorded was below a selected base discharge, and (2) recorded rainfall was not representative of the basin rainfall. The streamflow site on Munson Slough at Capital Circle (map number 7) was intended for only a daily discharge site and therefore not used in the calibration of the model.

The model calibration is accomplished in two steps. First, the seven parameters used to compute the volume of runoff are automatically adjusted until the difference between synthesized volumes and the observed volumes of runoff are minimized. The initial parameter values are determined from soil types, basin characteristics, and climatological factors.

As input to the soil-moisture and infiltration component, calibration of the model requires the following: unit and daily rainfall; unit discharge; daily evaporation; and impervious area as percentage of the total drainage area.

The method of determining optimum parameter values is based on an optimization technique by Rosenbrock (1960). The technique is a trial and error procedure. The model is programmed to change a parameter value and then recompute the objective function based on the new set of values. If an improvement is made, the set is retained; if not, the old set of values is retained. This process is followed for each parameter until improvement stops. The objective function is computed as the sum of the squared deviations of the logarithms of the difference between the synthesized flood volumes and the observed flood volumes.

In the second step, the volume parameters are held constant and the flow is routed to the outlet of the basin. A line printer plot is generated with the synthesized hydrograph overlaying the observed hydrograph. A visual comparison is made; if there is a significant difference, the parameter input values are checked and revised, and the calibration process is repeated until satisfactory results are obtained.

The results of the final calibrations are shown in table 3. Limits were placed on the volume parameters to conform to the range suggested by Lichty and Liscum (1978, p. 35). The allowable ranges for volume parameters are given in parentheses.

Flood-Peak Synthesis

Flood-peak synthesis is the process whereby flood discharge data are generated from long-term daily rainfall, daily evaporation, and unit rainfall, for the period of record, and calibrated model parameters for each site. The model generates flood hydrographs for each event entered for each rainfall-runoff site. Annual peak discharges are selected from the synthesized data.

The nearest long-term evaporation station is at Milton, Fla. Comparisons of available records indicate that daily evaporation does not vary greatly from Milton to Tallahassee. Also, the model is fairly insensitive to changes in evaporation. The National Weather Service recording rain gage, located initially at Thomasville, Ga., and later moved to Coolidge, Ga., is the nearest long-term station for which unit values are available. Based on information from National Weather Service (formerly U.S. Weather Bureau) Technical Report 40 (U.S. Department of Commerce, 1961), it was determined that some correction should be made to account for Tallahassee being nearer the coast than Thomasville-Coolidge. The nearest long-term rainfall record near the coast is at Pensacola, Fla. It was decided, therefore, to use the Thomasville-Coolidge and Pensacola unit rainfall records to generate two separate 60-year annual peak series for each gaging station for use in the flood-frequency analysis.

Table 3.--Calibrated model parameters

Infiltration component

PSP: in inches of pressure, the suction at the wetted front for soil moisture at field capacity (0.5 to 10).
KSAT: in inches per hour, the effective saturated value of hydraulic conductivity (0.01 to 0.5).
RGF: the ratio of the suction at the wetted front for soil moisture at wilting point to that at field capacity (1 to 45).

Antecedent moisture component

BMSM: in inches, the soil moisture storage at field capacity (1 to 12).
EVC: coefficient to convert pan evaporation to potential evapotranspiration (0.65 to 0.75).
RR: the percentage of daily rainfall that infiltrates into the soil (set at 0.85).
DRN: in inches per day, a coefficient controlling the rate of drainage of the infiltrated soil moisture (set at 1.0).

Routing component

KSW: in hours, time characteristic for linear reservoir routing.
TC: in minutes, length of the base of the translation hydrograph.

Flood events: number of floods used in calibration.

Standard error: standard error of simulated estimate.

Map location No.	Parameters PSP	KSAT	RGF	BMSM	EVC	KSW	TC	Number of floods	Standard error
6	6.43	0.260	33.8	4.61	0.735	15.4	163	15	19.1
8	9.84	.461	44.9	3.92	.704	1.95	31	31	25.5
9	9.80	.440	44.9	3.50	.745	1.30	50	14	20.9
10	9.62	.481	41.6	2.96	.740	.238	52	35	14.9
12	6.60	.120	42.0	8.00	.740	.900	53	39	24.3
13	7.80	.386	30.2	6.76	.743	1.10	26	27	28.4
16	4.08	.163	28.6	4.90	.714	6.00	80	9	22.5
17	4.13	.156	37.3	2.14	.731	.718	19	21	21.1
20	6.00	.270	44.0	7.00	.660	9.60	117	18	26
21	9.00	.418	44.2	8.16	.672	21.6	244	17	25.3
22	9.72	.468	43.2	3.15	.682	8.85	125	13	18.7
24	8.41	.156	14.7	5.20	.749	6.65	74	9	24.3
26	7.47	.470	23.8	2.69	.687	3.26	167	9	30.4
28	6.44	.365	12.0	2.17	.750	4.88	468	9	50.8
29	5.66	.434	44.6	5.41	.732	12.4	330	15	26.0

Flood-Frequency Analysis

The U.S. Water Resources Council (1981, p. 3) recommends the log-Pearson type III distribution for use as the base method for flood-frequency analysis. In this investigation, a log-Pearson distribution of the annual peak discharges, generated as described in the previous section, was made. The log-Pearson type III distribution is defined by three statistical parameters: the mean, the standard deviation, and the skew of the logarithms of the data. Station skew was used for all stations because regional skew is based on rural data and generally large drainage basins.

Flood magnitudes for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence intervals were determined for each station for both the Thomasville-Coolidge and Pensacola annual synthetic peak series. This resulted in two different frequency curves, one for each station. These frequency curves were then combined into a single frequency curve for each station by computing a weighted average using Technical Report 40 (U.S. Department of Commerce, 1961) as a guide. Flood magnitudes based on the Thomasville-Coolidge rainfall data were multiplied by 0.8, flood magnitudes based on the Pensacola rainfall data were multiplied by 0.2, and the results were summed to obtain the weighted flood magnitudes for each gaging station in Tallahassee. Table 4 gives the weighted synthetic flood magnitudes so computed. These were considered the best estimate of flood frequency for each site and were used in the regression analysis described in the next section of this report.

An alternate method of computing synthetic flood-frequency curves for a gaging station is described by Lichty and Liscum (1978). This method uses model calibration parameters in conjunction with climatic factors to develop station frequency curves. This eliminates the task of flood-peak synthesis and log-Pearson frequency analysis. Flood magnitudes for the 2-, 25-, and 100-year recurrence intervals were computed for all Tallahassee stations using the Lichty and Liscum (1978) map-model method. This method was used only for comparison and as an additional check on the magnitudes computed from the synthetic data. The flood magnitudes computed range around the station data. The maximum difference ranged from +39 to -30 percent for the 2-year flood and from +34 to -34 percent of the 100-year flood. Comparative results from the Lichty and Liscum method are given in Supplementary Data. Figures 5 and 6 show examples of two frequency curves, for each method, for northeast drainage ditch at Hadley Road (map number 26) and St. Augustine Branch at Wahnish Way (map number 13).

Regression Analysis

Because flood information is collected at only a few of the many sites where flood data are needed, the flood information must be extended from gaged to ungaged sites by regional analysis. Riggs (1973, p. 2) describes regression analysis as a useful regionalization method. Regression relates the discharge of a given flood frequency to basin characteristics. The regression model has the form:

$$Q_T = cA^aB^b \tag{1}$$

where

Q_T is the peak discharge for a T-year interval;
A and B are basin characteristics; and
a, b, and c are constants for recurrence interval T.

Multiple regression provides a mathematical relation between the dependent variable (flood magnitude) and the independent variables (basin characteristics) and a measure of accuracy of the relation. A measure of the usefulness of each independent variable in the relation is also defined.

Table 4.--Flood-frequency data for gaging stations

Map location No.	Discharge, in cubic feet per second						
	2-year recurrence interval Q_2	5-year recurrence interval Q_5	10-year recurrence interval Q_{10}	25-year recurrence interval Q_{25}	50-year recurrence interval Q_{50}	100-year recurrence interval Q_{100}	500-year recurrence interval Q_{500}
6	876	1,450	1,930	2,630	3,260	3,990	6,100
8	2,270	3,260	3,970	4,970	5,780	6,670	9,120
9	1,040	1,570	1,960	2,530	3,020	3,560	5,150
10	230	307	353	407	445	482	566
12	1,890	2,650	3,160	3,810	4,300	4,790	6,040
13	1,060	1,520	1,830	2,230	2,550	2,880	3,710
16	784	1,320	1,740	2,330	2,830	3,360	4,800
17	100	164	213	280	335	393	547
20	51	82	108	145	177	213	316
21	226	424	615	947	1,270	1,690	3,080
22	149	237	311	427	532	654	1,030
24	147	421	712	1,230	1,740	2,370	4,330
26	41	77	110	162	212	270	451
28	156	354	549	883	1,210	1,600	2,880
29	484	927	1,360	2,100	2,840	3,760	6,880

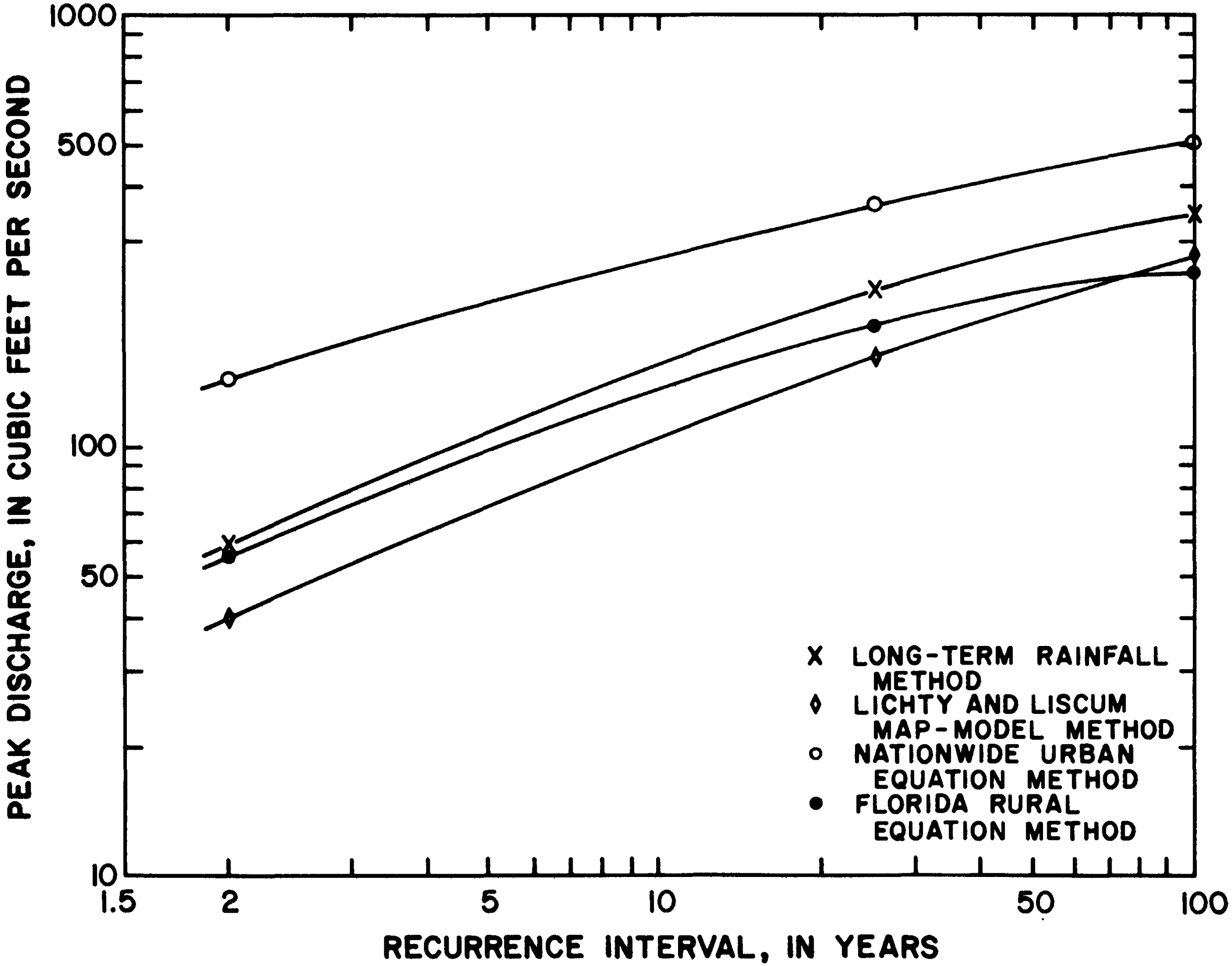

Figure 5.--Flood-frequency curves representing different estimating methods for northeast drainage ditch at Hadley Road (map number 26, Lake Lafayette basin).

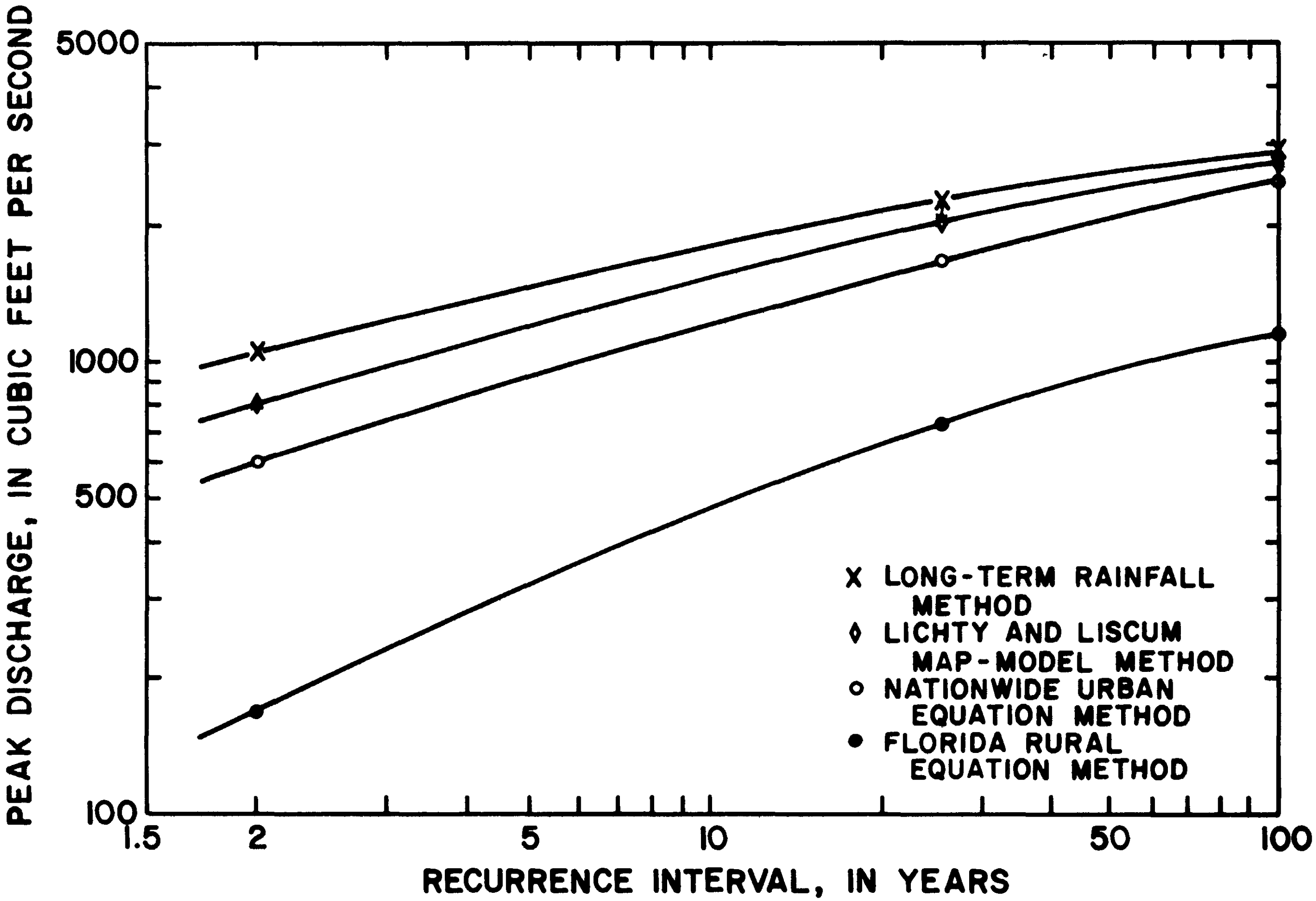

Figure 6.--Flood-frequency relations at St. Augustine Branch at Wahnish Way (map number 13).

Previous studies indicate that peak discharge is linearly related to basin characteristics if logarithmic transformations of each are used. Therefore, all discharge peaks and basin characteristics were transformed into logarithmic form before the regression was performed.

A data-analysis system called Statistical Analysis System (SAS)[1] was used to perform the multiple regression (Helwig and Council, eds., 1979, p. 392). SAS contains five methods of stepwise regression. The stepwise procedure "maximum R^2 improvement" (MAXR) was selected to determine which of the independent variables would be included in the regression model.

The square of the multiple correlation coefficient (R^2) measures how much variation in the dependent variable can be accounted for by the model. The MAXR method begins by finding the one-variable model producing the highest R^2 and adds another variable that will produce the largest increase in R^2. Each variable in the two-variable model is compared to each variable not in the model. MAXR determines if removing one variable and replacing it with another would improve R^2. Comparison or replacement of variables continues until the "best" two variable model, three-variable model, and so forth, is developed.

Long-term synthetic flood-frequency data from all of the gaging stations were used in the first regression analysis. These first attempts produced some unusual results indicating outliers in the data base. After careful analysis of the flood-frequency data, two stations with uncommonly steep frequency curves were removed from the analyses. These stations were the northeast drainage ditch at Capital Circle (map number 28) and Fords Arm Tributary (map number 24). The model calibration for these stations was not as good as others because data were limited to generally low peaks. This could explain why these stations were apparent outliers and would justify their removal.

A geographic bias was noted, as expected, in the Lake Lafayette basin when flood magnitude was plotted against drainage area. The Lake Lafayette basin has a significant amount of natural detention storage as compared to the more developed drainage pattern of the other basins in the study area. This storage results in smaller peak discharges for equivalent size drainage areas in other basins. Wilbert O. Thomas, Jr. (U.S. Geological Survey, Reston, Va., written commun., 1982) suggested use of a qualitative variable in accounting for a change in a characteristic used in the regression. A new "basin characteristic," or regression constant, to indicate location was created for regression analyses. All sites within Lake Lafayette basin were assigned a location value of one. All other sites were assigned a value of zero. The regression was repeated treating location as a basin characteristic except it cannot be transformed into log units. The resulting model had two constants which were combined to produce a constant for sites in the Lake Lafayette basin and a different constant for all other sites. Significant improvement in the regresssion resulted after this change was made. Drainage area, impervious area, and geographic location were the only basin characteristics significant at the 95 percent confidence level. All three were significant for all recurrence interval floods.

[1]The use of brand-named products in this report is for identification only and does not constitute endorsement by the U.S. Geological Survey.

The regression models for estimating the magnitude of the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence interval floods has the form:

$$Q_T = C\ DA^{x1} IA^{x2}; \qquad \text{or} \qquad (2)$$

$$Q_T = C_L DA^{x1} IA^{x2};$$

where

Q_T = the peak discharge for the desired recurrence interval flood, in cubic feet per second;
C = the regression constant for all sites outside the Lake Lafayette basin;
C_L = the regression constant for all sites in the Lake Lafayette basin;
DA = the contributing drainage area, in square miles; and
IA = the impervious area, in percentage, of the drainage area.

The coefficients for the regression models (2) for urban Leon County are summarized in table 5. The 2-year and 100-year station and regression flood magnitudes are compared in figures 7 and 8. These graphs illustrate close agreement throughout the range experienced. Supplementary Data contain comparative results for all recurrence intervals.

The equations presented in this report should be used for the developing area of Leon County. It is important to be aware that these equations are based on the range of values given previously (table 2). Extreme caution should be exercised in their use outside of that range.

Table 5.--Regression model coefficients for urban Leon County

Recurrence interval, T, in years	Exceedance probability	Regression constant		Exponents		R^2	Standard error of regression, in percent
		C	C_L	X_1 (DA)	X_2 (IA)		
2	0.5	10.7	1.71	0.766	1.07	0.99	18
5	.2	24.5	4.51	.770	.943	.98	18
10	.1	39.1	7.98	.776	.867	.98	20
25	.04	63.2	14.6	.787	.791	.98	22
50	.02	88.0	22.1	.797	.736	.97	24
100	.01	118	32.4	.808	.687	.97	25
500	.002	218	71.7	.834	.589	.97	30

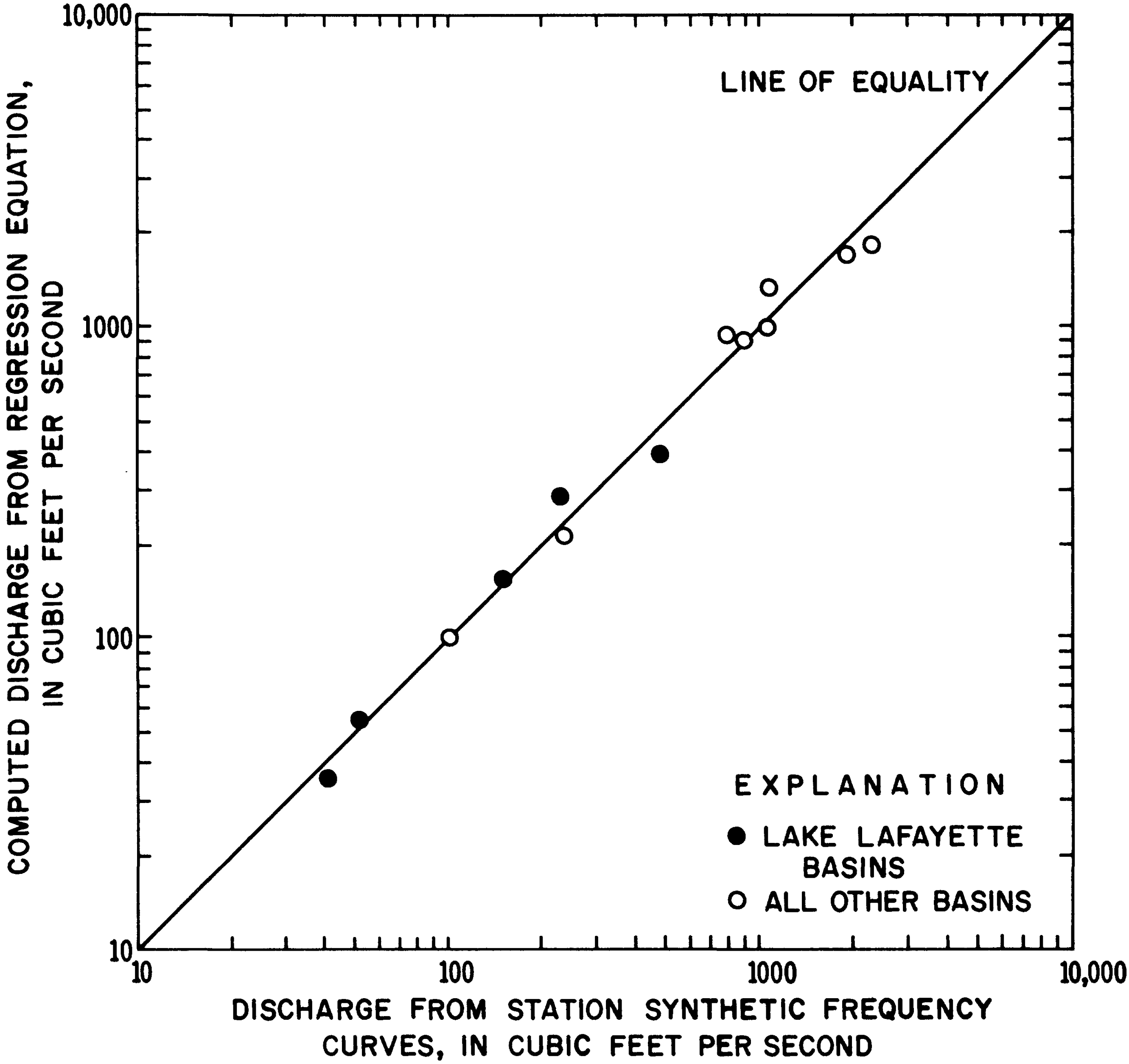

Figure 7.--Comparison of station and regression flood magnitudes for 2-year recurrence interval.

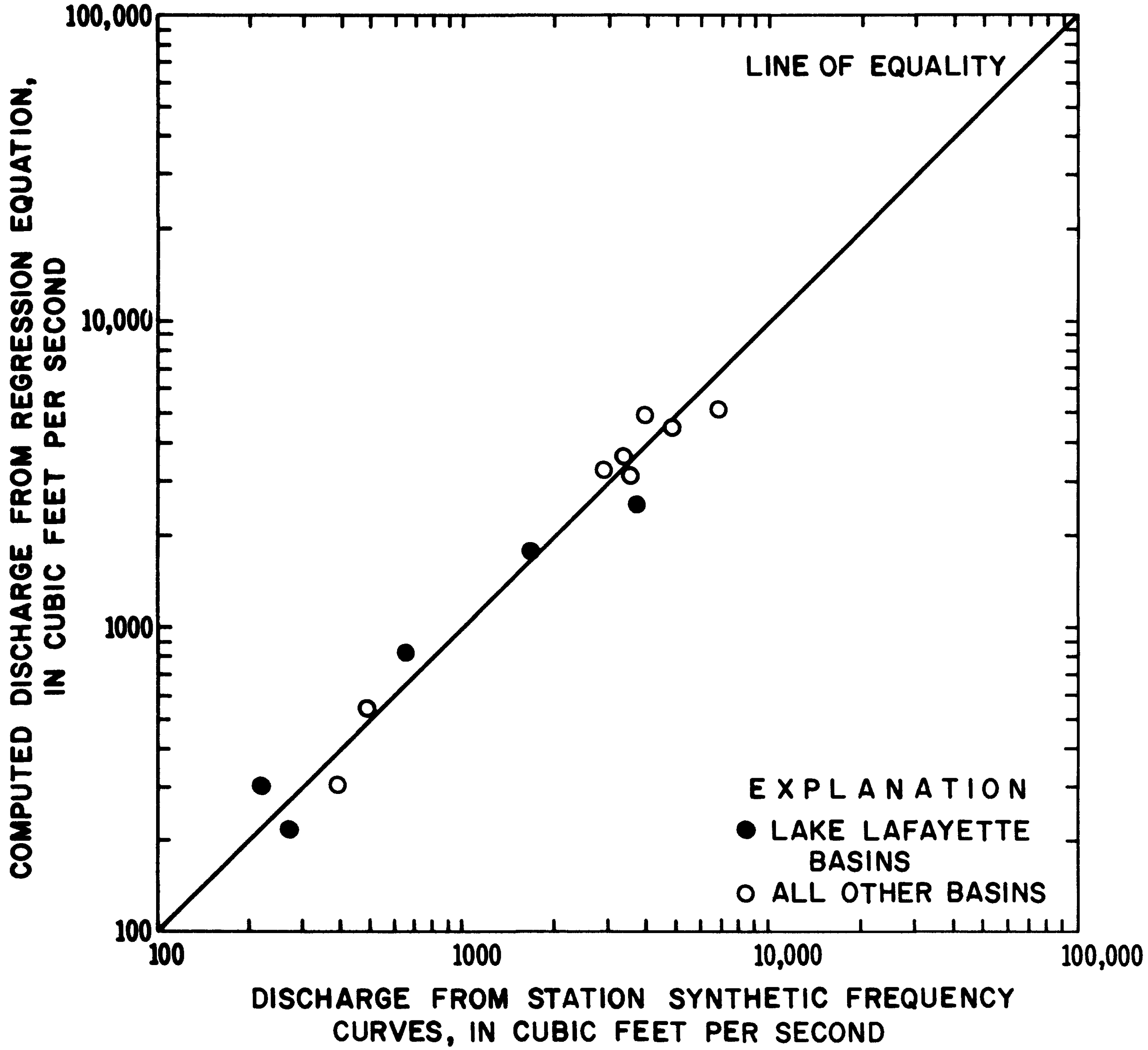

Figure 8.--Comparison of station and regression flood magnitudes for 100-year recurrence interval.

Accuracy of Regression

The accuracy or reliability of a regression equation can be expressed in two ways, first the standard error of prediction and second in the equivalent years of record.

The standard error of prediction is an approximation, in percent, of the ability of the regression equation to predict the magnitude of a given recurrence interval flood at any site, gaged or ungaged. Computation of the standard error of prediction (SEP) is described by Hardison (1971) and Inman (1983). The interstation correlation is a factor in computation of the SEP and was assumed to be 0.9, based on information from other studies. Other factors necessary for the computation of the SEP were the average standard deviation and the average skewness of the logarithms of the annual peak discharges. These factors were computed from the long-term synthesized data for each station and were 0.233 and 0.135, respectively. The number of stations used in the regression analysis was 13 and the number of independent variables was 3.

The equivalent years of synthesized record is also a factor in computation of the SEP. Because 60 years of synthetic data are not equivalent to 60 years of observed data, it was necessary to compute the equivalent length of synthesized data for this computation.

The equivalent years of record were computed from data given in a report by Lichty and Liscum (1978) in which they studied the synthesis of flood-peak data at 98 gaging stations in the eastern United States. It was assumed, for purposes of computing the equivalent length of synthesized data, that model error, or space-sampling error, is analogous to their average map-model error variance. This estimate of model error was converted to equivalent length of synthesized data by methods described by Hardison (1971). This method also requires an estimate of the average skewness and average standard deviation of the logarithms of peak discharges, which were assumed to be equal to the observed values given by Lichty and Liscum (1978).

Hardison (1971) defines equivalent years as the number of years of actual streamflow records at a site required to give the same accuracy as that obtained from the regression analysis.

Standard error of prediction, equivalent length of synthetic data, and accuracy of regression in equivalent years of record for each recurrence interval are given below.

Recurrence interval	Standard error of prediction, in percent	Equivalent length of synthetic record, in years	Accuracy of regression, in equivalent years of record
2	36	3	2
5	32	7	4
10	33	10	6
25	35	14	7
50	37	16	8
100	40	17	8
500	47	17	8

Probability of Exceedance

Bridges and culverts are designed for a specific recurrence interval flood magnitude. This does not preclude, however, a rare event occurring during the life of the structure. The U.S. Water Resources Council (1981, Appendix 10-1) defines risk as "the probability that one or more events will exceed a given flood magnitude within a specified period of years." Table 6 gives the percent chance of a flood for a given recurrence interval being exceeded during a given time period. For example, a 25-year flood has a 4 percent chance of exceedance in any 1-year period, a 64 percent chance of exceedance in any 25-year period, and a 98 percent chance of exceedance in any 100-year period.

Table 6.--Probability of a flood being exceeded during a given time period, in percent

Recurrence interval,	Period of time, in years					
T, in years	1	5	10	25	50	100
5	20	67	89	([1])	([1])	([1])
10	10	41	65	93	([1])	([1])
25	4	18	34	64	87	98
50	2	10	18	40	64	87
100	1	5	10	22	40	63

[1]Probability greater than 99 percent but less than 100 percent.

Comparison of Nationwide Urban Equations

Because of the limited data used in this study, a comparison was made between flood magnitudes computed using the local equations and equations presented by Sauer and others (1981) which are based on a much larger data base. These regression equations have the form:

$$UQ_T = cDA^{x1}SL^{x2}(RI2+3)^{x3}(ST+8)^{x4}(13-BDF)^{x5}IA^{x6}RQ_T{}^{x7} \qquad (3)$$

where

UQ_T = urban peak discharge for the recurrence interval, in cubic feet per second;
c = regression constant;
DA = contributing drainage area, in square miles;
SL = main channel slope, in feet per mile (if slope is greater than 70, use 70 in the equation);
RI2 = rainfall intensity, in inches, for the 2-hour, 2-year occurrence (for Tallahassee use 2.6);
ST = storage, in percentage, of the drainage area;
BDF = basin development factor;
IA = impervious area, in percentage, of the drainage area;
RQ_T = equivalent rural peak discharge, for the desired recurrence interval, as computed from equations presented by Bridges (1982, p. 9-11); and
x1,x2... = exponents of the regression.

Table 7.--Regression model coefficients for nationwide urban equation

	Recurrence interval, in years						
	2	5	10	25	50	100	500
Regression constant	2.35	2.70	2.99	2.78	2.67	2.50	2.27
Exponents							
X_1	.41	.35	.32	.31	.29	.29	.29
X_2	.17	.16	.15	.15.	.15	.15	.16
X_3	2.04	1.86	1.75	1.76	1.74	1.76	1.86
X_4	-.65	-.59	-.57	-.55	-.53	-.52	-.54
X_5	-.32	-.31	-.30	-.29	-.28	-.28	-.27
X_6	.15	.11	.09	.07	.06	.06	.05
X_7	.47	.54	.58	.60	.62	.63	.63
R^2	.93	.93	.93	.93	.92	.92	.90
Standard error, in percent.	38	37	38	40	42	44	49
Adjusted constants for Lake Lafayette basin.	.74	1.02	1.20	1.29	1.34	1.38	1.65

Table 7 summarizes the nationwide coefficients needed to compute the urban peak discharge using the above equation (3). In addition, this equation requires an estimate of the equivalent rural peak discharge at the urban site. The regression equation to compute the equivalent rural peak discharge (Bridges, 1982) has the form:

$$RQ_T = C\ DA^{B1}\ SL^{B2}\ (LK+3.0)^{B3} \tag{4}$$

where

RQ_T = rural peak discharge for a recurrence interval of T-years, in cubic feet per second;
C = regression constant;
DA = contributing drainage area, in square miles;
SL = channel slope, minimum value set at 0.9, in feet per mile (for example, if SL is less than 0.9 foot per mile, then SL = 0.9 foot per mile);
LK = surface area of lakes and ponds expressed as a percentage of the contributing drainage area; and
B1, B2, B3 = exponents of the regression.

Table 8 summarizes the coefficients for use in the rural equations (4) presented above.

Table 8.--Regression model coefficients for rural equations

[From Bridges, 1982, p. 11]

Recurrence interval, T, in years	Exceedance probability	Regression constant C	Regression exponents DA B_1	SL B_2	(LK+3) B_3	R^2	Standard error, in percent
2	0.5	58.9	0.824	0.387	-0.785	0.919	43.7
5	.2	117	.844	.482	-1.06	.905	45.9
10	.1	164	.860	.534	-1.21	.891	49.3
25	.04	234	.882	.586	-1.37	.870	54.8
50	.02	291	.900	.626	-1.48	.853	59.4
100	.01	351	.918	.658	-1.58	.835	64.6
200	.005	417	.936	.685	-1.67	.819	69.7
500	.002	507	.960	.725	-1.79	.797	76.5

A comparison of the Leon County station data with the nationwide equations was made and the results are shown in the Supplementary Data. A graphical comparison for the 2-year and 100-year results is shown in figures 9 and 10. These comparisons indicate reasonably good results for those sites not in the Lake Lafayette basin. For sites in the Lake Lafayette basin, where detention storage is significant, the nationwide equations overestimate the peaks. This is as expected because the nationwide equations were developed for sites where detention storage was not significant.

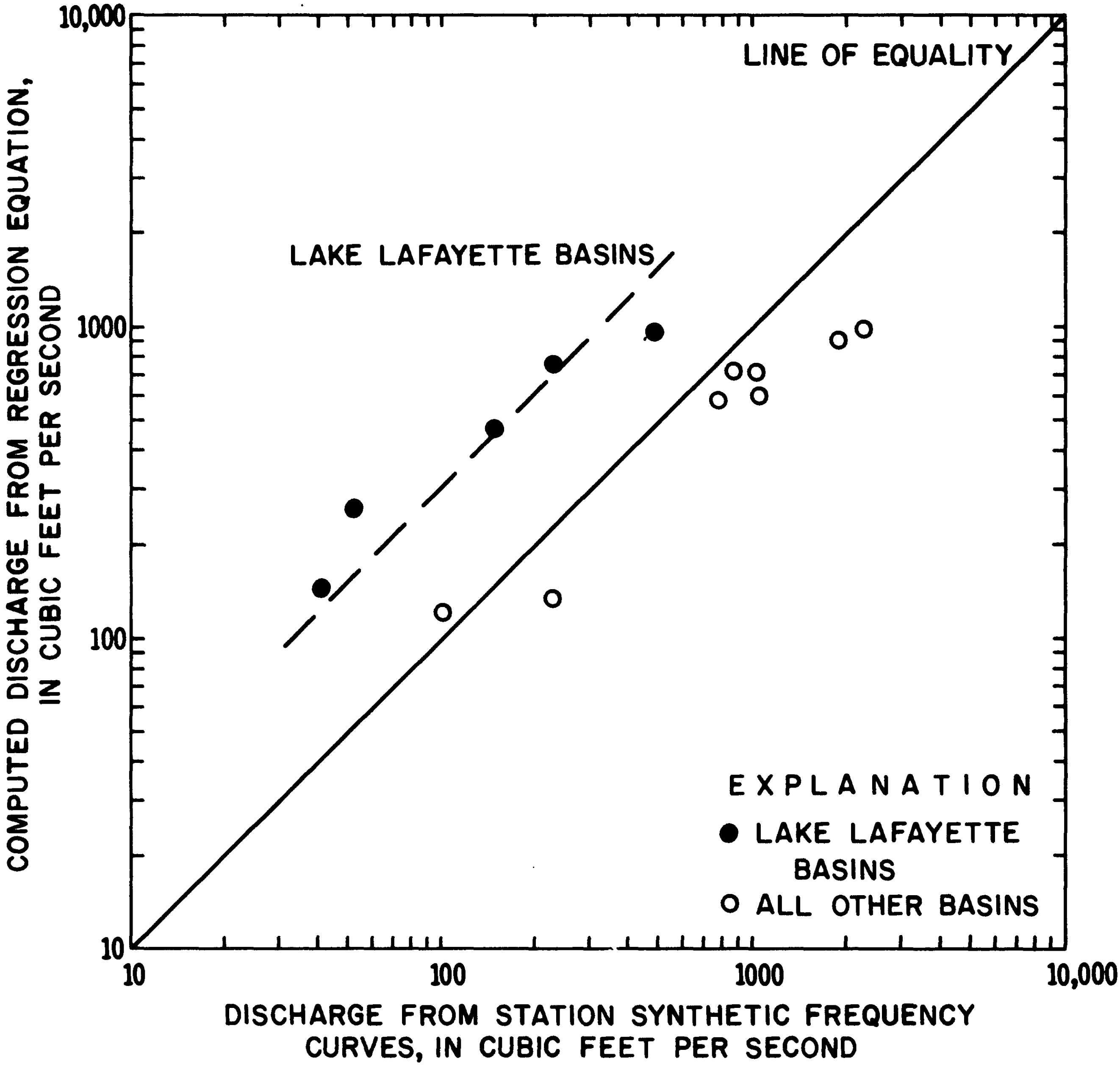

Figure 9.--Comparison of station and nationwide urban flood magnitudes for 2-year recurrence interval.

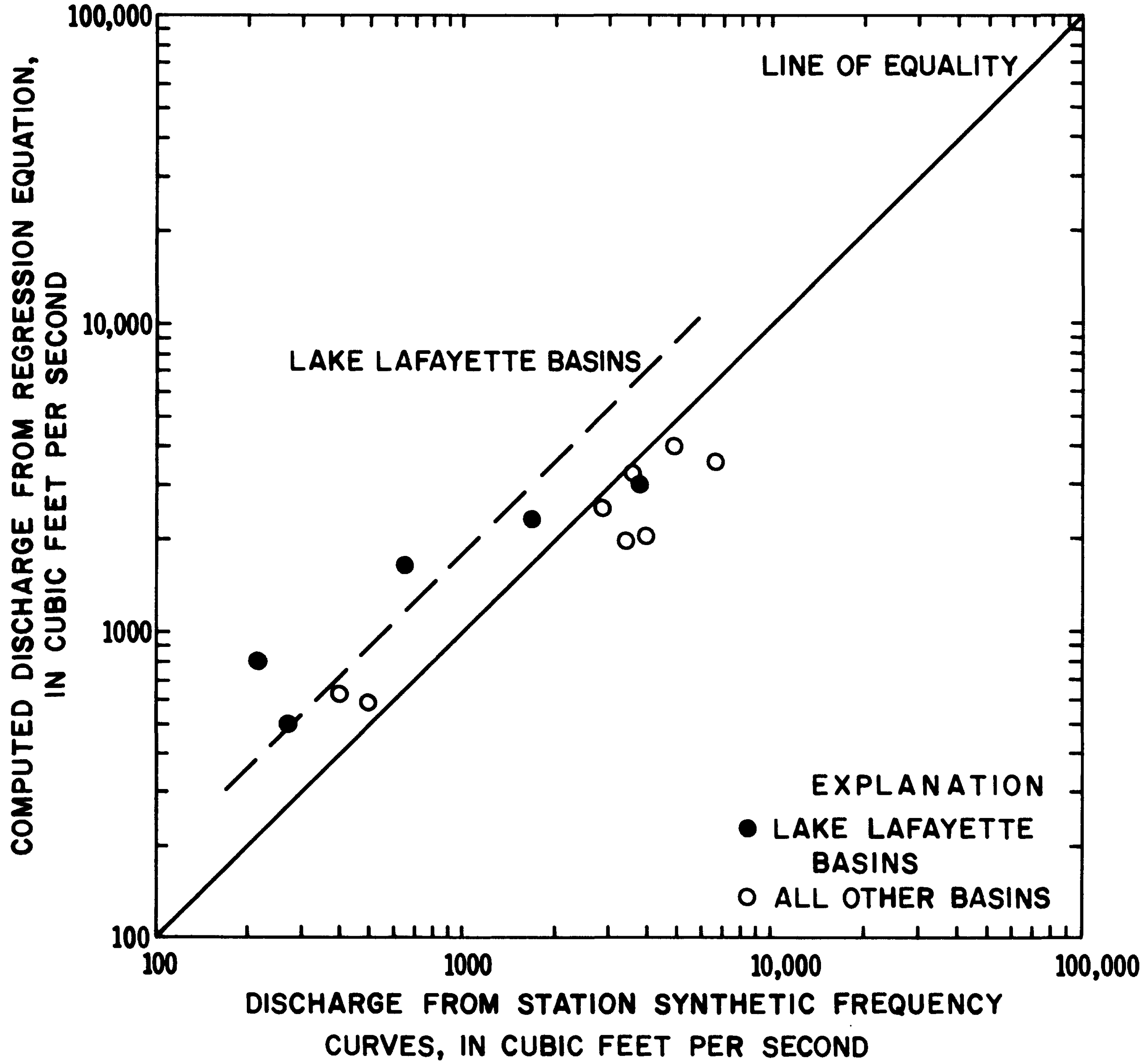

Figure 10.--Comparison of station and nationwide urban flood magnitudes for 100-year recurrence interval.

Adjustments were made to the nationwide equations as indicated by the dashed lines in figures 9 and 10. These adjustments provide a means of accounting for detention storage in the Lake Lafayette basin. The adjustment was made by reducing the equation constant, C, as shown in table 7. The nationwide equations, or the adjusted equations for Lake Lafayette basin, may be used when basin characteristics are outside the range of those used in this study.

APPLICATION OF TECHNIQUES

The regression equations presented can be used to estimate flood magnitudes for developing basins in Leon County. The procedure of weighting station flood magnitudes with regression values recommended by the U.S. Water Resources Council (1981) is not recommended for Leon County because of the short period of station data available. Flood magnitudes from regression equations should be used for all sites of interest.

A step-by-step procedure for determining flood magnitude for the desired recurrence interval flood is given in the example below:

Determine the flood magnitudes for St. Augustine Branch at Wahnish Way (map number 13).

1. Planimeter the contributing drainage area from U.S. Geological Survey topographic map ($DA = 2.06\ mi^2$).

2. Determine the percentage impervious area ($IA = 54$ percent).

3. The site is not in the Lake Lafayette basin; therefore, the equations for flood magnitudes are:

2-year flood magnitude

$$Q_2 = 10.7\ DA^{0.766}\ IA^{1.07}$$

$$Q_2 = 10.7(2.06)^{0.766}\ (54)^{1.07}$$

$$Q_2 = 1{,}330\ ft^3/s$$

5-year flood magnitude

$$Q_5 = 24.5\ DA^{0.770}\ IA^{0.943}$$

$$Q_5 = 24.5(2.06)^{0.770}\ (54)^{0.943}$$

$$Q_5 = 1{,}840\ ft^3/s$$

10-year flood magnitude

$$Q_{10} = 39.1\ DA^{0.776}\ IA^{0.867}$$

$$Q_{10} = 39.1(2.06)^{0.776}\ (54)^{0.867}$$

$$Q_{10} = 2{,}180\ ft^3/s$$

25-year flood magnitude

$$Q_{25} = 63.2\ DA^{0.787}\ IA^{0.791}$$

$$Q_{25} = 63.2(2.06)^{0.787}\ (54)^{0.791}$$

$$Q_{25} = 2,620\ ft^3/s$$

50-year flood magnitude

$$Q_{50} = 88.0\ DA^{0.797}\ IA^{0.736}$$

$$Q_{50} = 88.0(2.06)^{0.797}\ (54)^{0.736}$$

$$Q_{50} = 2,950\ ft^3/s$$

100-year flood magnitude

$$Q_{100} = 118\ DA^{0.808}\ IA^{0.687}$$

$$Q_{100} = 118(2.06)^{0.808}\ (54)^{0.687}$$

$$Q_{100} = 3,280\ ft^3/s$$

500-year flood magnitude

$$Q_{500} = 218\ DA^{0.834}\ IA^{0.589}$$

$$Q_{500} = 218(2.06)^{0.834}\ (54)^{0.589}$$

$$Q_{500} = 4,170\ ft^3/s$$

SUMMARY

A U.S. Geological Survey urban rainfall-runoff model was calibrated for each of 15 gaging stations in Leon County, Fla. Calibrations for two stations were questionable and those stations were deleted from the regional analysis. The calibrated models were used to generate long-term synthetic flood-peak records from Thomasville-Coolidge, Ga., and Pensacola, Fla., rainfall records. Flood frequency curves were developed based on the data for each rainfall station and then by giving a weight of 0.2 for Pensacola, Fla., and 0.8 for Thomasville-Coolidge, Ga., frequency curves for Tallahassee were constructed.

Flood-frequency analysis developed from the synthetic flood-peak record and measured basin characteristics were used in multiple linear regression analyses to derive regional flood-frequency equations. These equations can be used to estimate flood magnitudes in Leon County for recurrence intervals from 2 to 500 years. The standard errors of regression range from ±18 percent to ±30 percent, and the standard errors of prediction range from ±32 percent to ±47 percent. A comparison of the Leon County station data to the nationwide equations indicate reasonably good results for thóse sites not in the Lake Lafayette basin,. For sites in the Lake Lafayette basin, where detention storage is significant, the nationwide equations overestimated the peaks.

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SUPPLEMENTARY DATA

Comparison of station, regression, nationwide urban (with adjustments for Lake Lafayette basin), and Lichty and Liscum (2-, 25-, and 100-year only) flood discharges.

Map location No.	Recurrence interval (year)	Flood discharges, in ft^3/s			
		Station	Leon County regression	Nationwide urban	Lichty and Liscum
6	2	876	912	706	685
	5	1,450	1,600	985	
	10	1,930	2,220	1,250	
	25	2,630	3,110	1,540	2,230
	50	3,260	3,950	1,790	
	100	3,990	4,910	2,050	3,160
	500	6,100	7,880	2,500	
8	2	2,270	1,810	975	1,780
	5	3,260	2,750	1,460	
	10	3,970	3,460	1,940	
	25	4,970	4,450	2,490	4,910
	50	5,780	5,280	3,010	
	100	6,670	6,160	3,580	6,670
	500	9,120	8,700	4,690	
9	2	1,040	986	727	786
	5	1,570	1,480	1,180	
	10	1,960	1,850	1,640	
	25	2,530	2,350	2,180	2,330
	50	3,020	2,760	2,740	
	100	3,560	3,180	3,320	3,290
	500	5,150	4,380	4,550	
10	2	230	213	137	173
	5	307	301	225	
	10	353	358	309	
	25	407	429	400	418
	50	445	479	507	
	100	482	526	591	550
	500	566	650	785	
12	2	1,890	1,700	914	1,450
	5	2,650	2,380	1,460	
	10	3,160	2,850	2,010	
	25	3,810	3,480	2,650	3,490
	50	4,300	3,960	3,320	
	100	4,790	4,450	4,040	4,530
	500	6,040	5,790	5,520	

Map location No.	Recurrence interval (year)	Flood discharges, in ft^3/s			
		Station	Leon County regression	Nationwide urban	Lichty and Liscum
13	2	1,060	1,330	601	814
	5	1,520	1,840	947	
	10	1,830	2,180	1,290	
	25	2,230	2,620	1,680	2,040
	50	2,550	2,950	2,080	
	100	2,880	3,280	2,500	2,710
	500	3,710	4,170	3,350	
16	2	784	940	574	550
	5	1,320	1,480	852	
	10	1,740	1,910	1,120	
	25	2,330	2,510	1,420	1,620
	50	2,830	3,020	1,700	
	100	3,360	3,560	1,990	2,220
	500	4,800	5,130	2,550	
17	2	100	101	121	99
	5	164	153	215	
	10	213	190	307	
	25	280	236	412	270
	50	335	271	531	
	100	393	305	633	361
	500	547	395	865	
20[1]	2	51	55	70	64
	5	82	97	129	
	10	108	134	180	
	25	145	192	263	203
	50	177	243	347	
	100	213	305	441	286
	500	316	493	740	
21[1]	2	226	286	241	228
	5	424	510	410	
	10	615	721	558	
	25	947	1,060	805	904
	50	1,270	1,390	1,030	
	100	1,690	1,790	1,310	1,370
	500	3,080	3,080	2,170	

[1]Lake Lafayette basin: Nationwide urban results are based on adjusted equations.

Map location No.	Recurrence interval (year)	Flood discharges, in ft³/s: Station	Leon County regression	Nationwide urban	Lichty and Liscum
22[1]	2	149	154	147	140
	5	237	264	263	
	10	311	362	367	
	25	427	516	536	507
	50	532	653	700	
	100	654	819	900	756
	500	1,030	1,330	1,520	
26[1]	2	41	36	46	57
	5	77	64	83	
	10	110	90	116	
	25	162	131	169	230
	50	212	168	221	
	100	270	212	279	345
	500	451	347	464	
29[1]	2	484	385	309	537
	5	927	693	519	
	10	1,360	986	706	
	25	2,100	1,470	1,020	2,060
	50	2,840	1,930	1,290	
	100	3,760	2,510	1,660	3,070
	500	6,880	4,400	2,740	

[1] Lake Lafayette basin: Nationwide urban results are based on adjusted equations.