PRELIMINARY EVALUATION OF FLOOD FREQUENCY RELATIONS IN THE URBAN AREAS OF MEMPHIS, TENNESSEE

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PREPARED IN COOPERATION WITH
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The following factors may be used to convert the English units published in this report to the International System of Units (SI):

Multiply English units	By	To obtain SI units
Feet (ft)	0.3048	Meter (m)
Square miles (mi^2)	2.590	Square kilometers (km^2)
Miles (mi)	1.609	Kilometers (km)
Acres	0.004047	Square kilometers (km^2)
Cubic feet per second (ft^3/s)	0.02832	Cubic meters per second (m^3/s)
Inches (in)	25.40	Millimeters (mm)
Cubic feet per second per acre ($(ft^3/s)/acre$)	6.9978	Cubic meters per second per square kilometer ($(m^3/s)/km^2$)

PRELIMINARY EVALUATION OF FLOOD FREQUENCY RELATIONS IN THE URBAN AREAS OF MEMPHIS, TENNESSEE

By

Charles W. Boning

U. S. Geological Survey

ABSTRACT

A storm-runoff relation for streams in the urban areas of Memphis was determined by a statistical evaluation of 59 flood discharges from 19 gaging stations. These flood discharges were related to drainage area, percent imperviousness of the drainage basin, and rainfall occuring over 120-minute periods. The defined relation is

$$Q = 158.3A^{.777A^{-.02}}(IMP+1)^{.227}(I120)^{.539(I120)^{.40}}$$

where Q is flood discharge in cfs, A is drainage area in square miles, IMP is percent imperviousness in the basin, and I120 is rainfall in inches, over 120 minute time period.

The defined relation was used to synthesize sets of annual flood peaks for drainage basins ranging from .05 square miles to 10 square miles and imperviousness ranging from 0 to 80 percent for the period of rainfall record at Memphis. From these series of flood peaks, frequency relations were defined and presented for 2, 5, 10, 25, 50 and 100 year recurrent intervals.

INTRODUCTION

Physical Setting

Memphis is in southwestern Tennessee, and borders the Mississippi River. The city was founded on the bluffs overlooking the river well above the dangers of flooding during high water. As the city grew, however, it expanded to the flood plains of smaller streams in the vicinity creating a flood hazard for residents or commercial establishments who chose to build in those areas.

The climate of Memphis is generally temperate. Summers are hot and winters are relatively mild, although below freezing temperatures are common in the winter for short periods. Average rainfall is about 47 inches per year. Although widespread flooding is most likely to occur from backwater from the Mississippi River or from flood water of the three principal streams entering the Mississippi River in the vicinity of Memphis, severe localized flooding for short duration is a threat from the smaller streams. This flooding is caused by the intense storms that are common to the area in the early spring months, and by severe thunderstorms that occur during summer months.

Natural stream channels in the Memphis area are nearly nonexistent. During initial stages of development most of the streams were dredged and straightened to lessen flood potential. As development intensified, the channels were generally lined, many years ago, with hand-placed rock and mortar and, more recently, by rectangular concrete canal-type structures. These improvements increase the carrying capacities of the channels and generally reduce the flood potential. Flooding however still occurs, particularly from those streams that drain highly industrialized areas where infiltration is greatly reduced and channel improvements and storm sewer networks shorten storm runoff time.

Purpose and Scope of the Project

In 1974 the U. S. Geological Survey, in cooperation with the Chickasaw Basin Authority, initiated a project to provide a data base for updating existing storm drainage design criteria, or to develop such criteria for areas where data are non-existent or inapplicable. This project developed through recognition by city and county officials of the lack of data adequate for the design of most efficient storm drainage facilities in the Memphis area. The investigation is presently being conducted through a cooperative agreement among the Geological Survey, the City of Memphis, and Shelby County, Tennessee.

The investigative procedure that was anticipated when the project was initiated may be summarized as follows:

1. Collect, using a network of rainfall and stream gages, a sufficient amount of data to define relationships between rainfall characteristics (amounts, durations, and intensities) and runoff characteristics (peak discharge, time required for runoff, and runoff volumes).

2. Simulate annual peak flows since 1900 using the rainfall data record of the National Weather Service and a computer model calibrated with data collected during this project.

3. Determine the flood frequency characteristics at each site using the simulated annual peak flows. This process will define the magnitude of the flood flows expected, on the average, every 50 years or every 100 years.

4. Compute or assign numerical values to stream basin or channel parameters that affect flood flows. Included in this set of parameters are characteristics such as drainage area, impervious area, storm sewer development, stream slope, soil index, and vegetative cover.

5. Define the effects that each significant basin parameter exhibits in controlling peak flows and runoff volumes. The method of defining these effects is best accomplished by the mathematical process known as "regression analysis."

6. Define the effects of urbanization on other runoff characteristics such as peak-flow lag times and the low-flow durations and amounts.

The ultimate objective of this project is to provide a comprehensive report containing mathematical equations, data tables, graphs, and nomographs that can be used by local government agencies, engineers, and developers to plan and design urban storm drainage structures or to monitor the construction of these facilities.

The rapid development of suburban Memphis dictates that the goals and objectives of the project be achieved as quickly as possible. Experience by local officials in design and maintenance of storm drainage improvements, indicates that the present design criteria are inadequate for most urban channel improvements, and that more appropriate design criteria be developed immediately.

The proposed analytic method for satisfying the goals of the project, that is, the calibration of a parametric rainfall-runoff model for each of many streams in the area, will unfortunately require several additional years of data collection. The vital need for interim design criteria that can be used in the current development of Memphis and its suburban areas dictates that alternative analytic techniques, using data collected thus far, be investigated.

This report describes an alternative approach to developing flood frequency characteristics, and presents results of the application of that technique to small streams in the urban and suburban areas of Memphis, Tennessee. The tool used is a regression model, with variations. The use of a parametric rainfall runoff model is not attempted in the analysis.

THE GAGE NETWORK

The rainfall and streamflow data collection network for the project was established in 1974 and 1975 and includes 29 stream gaging stations and 34 recording rain gages. In 1976 one stream gage and one rain gage were relocated. The complete network is shown in figure 1 and the station names and site reference numbers are listed in Tables 1 and 2. Also included in figure 1 are gaging stations on Nonconnah Creek, Wolf River, and Loosahatchie River, the principal tributaries to the Mississippi River in the Memphis area. These three stations were put into operation in 1969, prior to the beginning of the present study.

Twenty of the stream gages are operated as flood hydrograph stations; that is they provide storm runoff information only. The remaining nine project stations are being operated as continuous record stations, and provide, in addition to storm runoff data, a full range of flow information. A rain gage is located either at each stream gage site or at a more advantageous place within the drainage basin of the stream. Five additional rain gages are situated about the periphery of the Memphis area to provide supplementary information about the general distribution of rainfall.

The gage network was designed to include basins that contain a wide range in the degree of urban development. Fourteen of the selected basins contain fully or nearly fully developed residential, commercial, or industrial areas, and little change is expected to occur in these basins during the next five to ten years. Nine basins are undergoing considerable modification as urbanization continues to expand to the outlying areas. The remaining six basins of the network are primarily rural and no significant change is expected in these basins during the next several years.

Another major consideration in the selection of streams for the gage network was to sample a range of drainage basin size. The need for stream-flow data from small basins dictated the search for streams in the Memphis area that would provide such information. This search was not as fruitful as desired owing to the general practice of using underground sewers to service small drainages. Nevertheless, streams draining basins as small as .05 square miles were found suitable for gaging purposes.

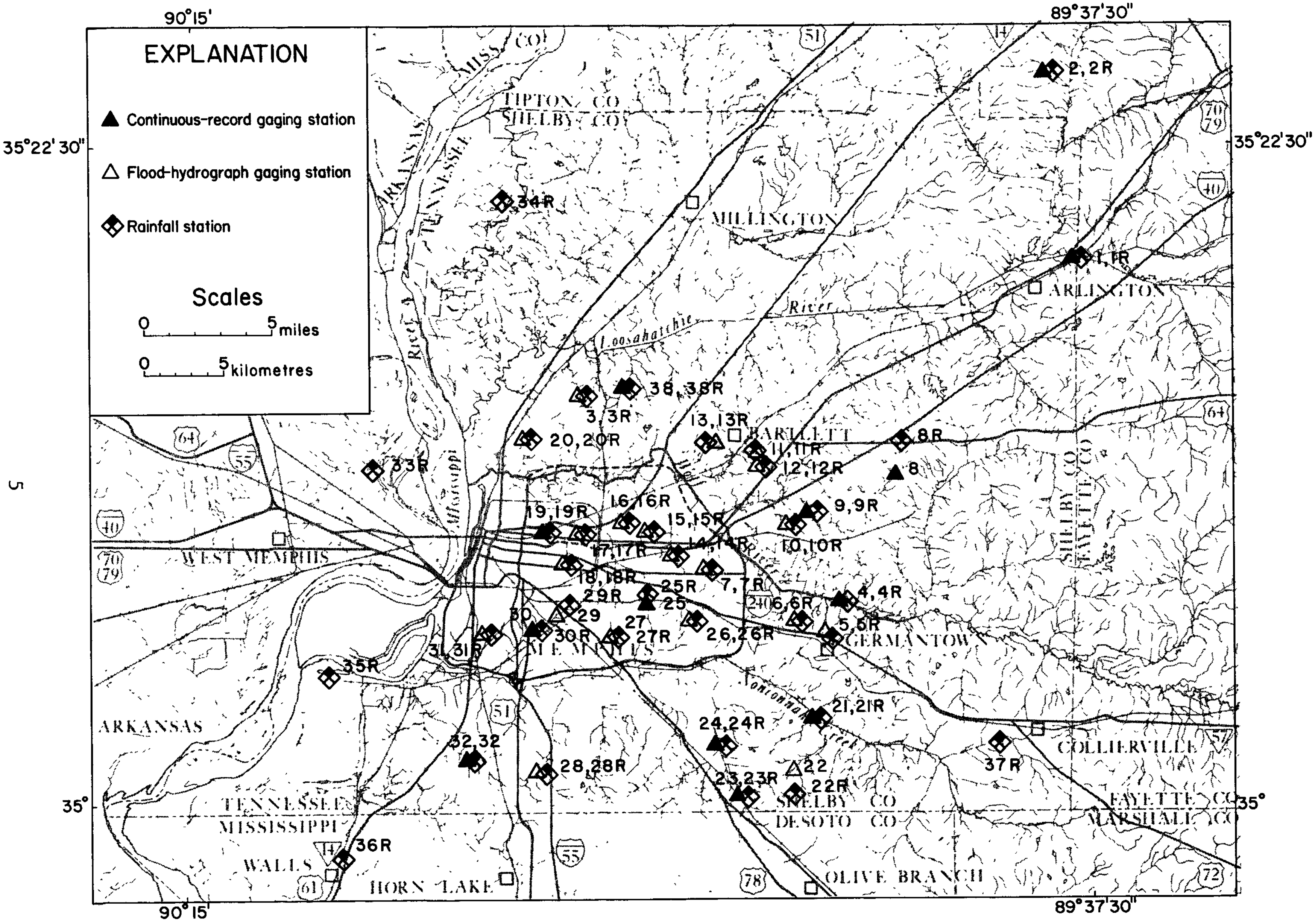


Figure 1.--Rain and stream gage network.

Table 1.--Stream gaging stations in Memphis metropolitan area

Map No.	Gaging Station No.	Name
1	07030240	Loosahatchie River near Arlington
2	07030245	Kelly Branch near Clopton (Discontinued)
3	07030300	Loosahatchie River tributary at St. Elmo Avenue at Memphis
4	07031650	Wolf River near Germantown
5	07031653	Wolf River tributary at Willey Road, at Germantown
6	07031657	Wolf River tributary at Noshoba Road at Germantown
7	07031665	White Station Creek at Rich Road, at Memphis
8	07031680	Fletcher Creek near Cordova
9	07031685	Fletcher Creek tributary at Charles Bryan Road, near Cordova
10	07031690	Fletcher Creek tributary at Whitten Road, at Memphis
11	07031694	Harrington Creek tributary at Elmore Park Road, at Bartlett
12	07031695	Harrington Creek tributary, at Hawthorne Road, at Bartlett
13	07031697	Harrington Creek tributary at Stage Road, at Bartlett
14	07031710	Harrison Creek at Charleswood Road, at Memphis
15	07031725	Workhouse Bayou tributary at Isabelle Street, at Memphis
16	07031730	Workhouse Bayou at Holmes Street, at Memphis
17	07031765	Overton Bayou at North Drive, at Memphis
18	07031773	Lick Creek at Jefferson Avenue, at Memphis
19	07031777	Lick Creek at Dickinson Street, at Memphis
20	07031795	Wolf River tributary at Whitney Avenue, at Memphis
21	07032195	Nonconnah Creek tributary at Shelby Drive, near Memphis
22	07032200	Nonconnah Creek near Germantown
23	07032222	Johns Creek tributary at Holmes Road, near Memphis
24	07032224	Johns Creek at Raines Road, at Memphis
25	07032241	Black Bayou at Southern Avenue, at Memphis
26	07032242	Cherry Bayou at Park Avenue, at Memphis
27	07032244	Cherokee Creek at Kimball Avenue, at Memphis
28	07032246	Days Creek at Shelby Drive, at Memphis
29	07032247	Parkway Bayou at South Parkway East, at Memphis
30	07032248	Cane Creek at East Person Avenue, at Memphis
31	07032249	Latham Branch at Valley Boulevard, at Memphis
32	07032260	Cypress Creek at Neely Road, at Memphis
38	07030295	Loosahatchie River tributary at New Allen Road, Memphis

Other factors that were recognized as being probably significant in affecting runoff characteristics are the extent and development of storm sewers, the density of storm sewer inlets, the channel or basin slope, and basin shape. In the design of the gage network, less consideration was given to these factors than to drainage size and urbanization. The number of gages installed for data collection and their area distribution, however, provides for a sampling of adequate ranges of these characteristics.

Table 2.--Rainfall stations in Memphis metropolitan area

Map No.	Latitude	Longitude	Name
1R	35°18'45"	89°38'19'	U.S. Highway 78, Arlington
2R	35°25'57"	89°39'15"	U.S. Highway 78, Clopton (Discontinued)
3R	35°13'56"	89°58'51"	Brookmead Avenue Memphis
4R	35°06'59"	89°48'05"	Germantown Road, Germantown
5R	35°05'37"	89°48'35"	Phillips 66 Station, U.S. 72, Germantown
6R	35°06'21"	89°49'54"	Neshoba Road, Germantown
7R	35°08'09"	89°53'37"	Rich Road, Memphis
8R	35°12'20"	89°45'22"	Sunset Church at U.S. Highway 64, Cordova
9R	35°10'07"	89°49'27"	Charles Bryan Road, Cordova
10R	35°09'27"	89°50'09"	Shelby Penal Farm, Memphis
11R	35°12'17"	89°51'30"	Quick Shop Grocery, Highway 64, Bartlett
12R	35°11'38"	89°51'02"	Elmore Park Civic Center, Bartlett
13R	35°12'20"	89°53'03"	Stage Road, Bartlett
14R	35°08'34"	89°55'00"	Charleswood Road, Memphis
15R	35°09'24"	89°56'04"	Isabelle Street, Memphis
16R	35°09'34"	89°57'04"	Holmes Street, Memphis
17R	35°09'14"	89°58'45"	Faith Temple, Trezevant Street, Memphis
18R	35°08'20"	89°59'30"	Jefferson Avenue, Memphis
19R	35°09'24"	90°00'14"	Dickinson Street, Memphis
20R	35°12'32"	90°01'16"	Whitney Avenue, Memphis
21R	35°02'59"	89°49'08"	Winchester Road near Germantown
22R	35°00'24"	89°50'01"	Thunder Hill Ranch at Holmes Road, Germantown
23R	35°00'20"	89°52'16"	Holmes Road, Memphis
24R	35°02'06"	89°53'10"	Raines Road, Memphis
25R	35°07'20"	89°56'04"	Memphis State University, Engineering Building
26R	35°06'24"	89°54'13"	Park Avenue, Memphis
27R	35°05'43"	89°57'31"	Kimball Avenue, Memphis
28R	35°01'00"	90°00'44"	Butterworth Road, Memphis
29R	35°06'52"	89°59'06"	South Parkway, Memphis
30R	35°06'02"	90°00'43"	East Person Avenue, Memphis
31R	35°05'56"	90°02'43"	Valley Boulevard, Memphis
32R	35°01'36"	90°03'23"	Neely Road, Memphis
33R	39°11'25"	90°07'19"	St. Francis Levee, Mound City, Ark.
34R	35°20'39"	90°01'55"	Meeman - Shelby Forest State Park
35R	35°04'30"	90°09'00"	Allen Steam Plant, Memphis
36R	34°58'19"	90°08'39"	Mississippi Weigh Station, Walls, Miss.
37R	35°02'15"	89°41'20"	Byhalia Road, Collierville
38R	35°11'16"	89°50'09"	New Allen Road, Memphis

DATA AVAILABLE

Streamflow and Rainfall Data

The data used in this analysis were collected from late in 1974, at the earliest installed rainfall and stage stations of the gage network, to late in 1976. The low incidence of storms during the latter year hampered the accumulation of data sufficient for modeling applications.

For the data collected at a gaging station to be useful they must satisfy several criteria. The actual data recorded at a stream flow station are stage data, or elevations of the water surface above an arbitrary datum. These data alone cannot be used as input to the selected analytic technique; corresponding values of flow or discharge must be determined. The data provided by the gaging stations were therefore further limited by the number of streams for which adequate stage-discharge relations have been defined. Of the 29 stations in the gage network, 19 possess stage-discharge relations that are considered sufficiently defined for use in modeling. These relations included many that were defined entirely by actual measurements of flow at various stages and some that were defined by theoretical equations of flow, and adjusted to one or two discharge measurements. Records for only a very few gaging stations that have entirely theoretical ratings were used in the analysis. These stations are located on streams having channel characteristics that generally yield reasonably accurate theoretical stage discharge relations.

For a storm and the resulting flow to be used as reliable input to a model, recorded precipitation must be representative of storm rainfall over the entire basin. Localized storms often produce rainfall with significant variations in intensity and amount in the basins, resulting in recorded rainfall either greater or less than the average in the basin. The available data were carefully screened by comparing the recorded rainfall at each site with that recorded at nearby gages, and those rainfall data that appeared to be either non-representative of rainfall over the entire basin or inaccurate were eliminated.

From an initial selected set of about 75 data values, the screening of rainfall reduced the data suitable for modeling input to 59 flood peaks and associated rainfall. These values were used in calibrating the regression model. As more data became available, either through the occurrence of additional storms or through the defining of additional stage-discharge relations, they were added to a second set of data that would test the calibrated model for accuracy and applicability.

The rainfall and streamflow data are collected at 5-minute increments of time and would be extremely voluminous if presented in tabular form. The severity of storms, however, can be aptly shown by a presentation of storm intensity. The 59 storms, therefore, are described by showing the maximum rainfall amounts that fell during selected time intervals of 5, 10, 30, 60, 120, and 180 minutes. These data, along with the peak discharges caused by the storms, were used as model input and are given in table 3.

Drainage Basin Characteristics

The independent variables selected for evaluation and for possible inclusion in the developed relations are described in the following paragraphs. These physical characteristics that influence streamflow are expressed by simplified representative indices, and are considered independent variables in the statistical process used. These variables do not comprise all of the factors that influence the runoff characteristics; only those that intuitively are considered to be highly significant are discussed. The values of the indices of the described characteristics are given in table 3 along with rainfall and peak runoff for the selected storms.

Drainage area -

Drainage area, A, has been shown in many studies to be the most significant parameter related to flood discharge for a given storm. The drainage areas for basins selected for the project range in size from 19.4 square miles to .05 square miles.

Urbanization -

It is generally accepted that urbanization causes increases in flood magnitudes and runoff volumes in those areas. One urbanization characteristic that can be measured and that has a significant effect on the rapidity and volume of storm runoff is the imperviousness, IMP, of the basin. Imperviousness is referred to as the percent of a basin that is covered by man-made structures, roads, and parking lots thus reducing infiltration from its normal capability to zero where those modifications exist. In studies in other urban areas, imperviousness has been found to be second only to drainage area as the most significant parameter or independent variable in regression analysis dealing with flood frequency (Dempster 1974).

In Memphis, the commercial and high-density residential areas in the basins drained by Lick Creek at Jefferson Street, site 18 and Overton Bayou at North Drive, site 17, have impervious areas covering more than 50 percent of the basins. The low-density residential areas characteristic of basins such as those drained by Harrington Creek tributary at Hawthorne Road, site 12,

Table 3.--Data matrix used

Stream Site	Storm Date	Peak Discharge (ft^3/s)	Drainage Area (mi^2)	Stream Slope (ft/ft)
5	Jul 2, 1975	199	0.21	.01300
5	Jul 23, 1975	78	0.21	.01300
7	Mar 28, 1975	650	2.45	.00672
7	Apr 9, 1975	790	2.45	.00672
7	Mar 29, 1976	980	2.45	.00672
8	Mar 28, 1975	300	1.45	.00551
8	Jul 23, 1975	310	1.45	.00551
9	Dec 24, 1974	680	3.18	.00419
9	Mar 12, 1975	760	3.18	.00419
9	Mar 28, 1975	620	3.18	.00419
10	Jan 10, 1975	89	.54	.00817
10	Mar 12, 1975	203	.54	.00817
10	Apr 9, 1975	101	.54	.00817
11	Dec 24, 1974	96	.33	.00833
11	Mar 28, 1975	180	.33	.00833
11	Jul 23, 1975	170	.33	.00833
12	Dec 24, 1974	80	.21	.01419
12	Jan 10, 1975	72	.21	.01419
12	Feb 23, 1975	85	.21	.01419
12	Mar 28, 1975	80	.21	.01419
12	Jul 23, 1975	180	.21	.01419
12	Mar 29, 1976	84	.21	.01419
14	Mar 28, 1975	760	1.59	.00548
14	May 26, 1975	300	1.59	.00548
14	Jul 23, 1975	480	1.59	.00548
15	Jul 23, 1975	45	.09	.00654
15	Nov 30, 1975	58	.09	.00654
16	Dec 24, 1974	440	1.30	.00510
16	Feb 5, 1976	415	1.30	.00510
16	Feb 17, 1976	350	1.30	.00510
16	Mar 29, 1976	480	1.30	.00510
17	Mar 28, 1975	197	0.30	.00946
17	Aug 30, 1975	115	0.30	.00946
17	Sep 9, 1975	123	0.30	.00946
17	Mar 29, 1976	150	0.30	.00946
18	May 11, 1975	590	1.00	.00658
19	Mar 12, 1975	1130	2.96	.00416
19	Mar 28, 1975	1110	2.96	.00416
19	Feb 5, 1976	660	2.96	.00416
19	Mar 29, 1976	705	2.96	.00416
21	Feb 5, 1976	390	1.58	.00602
21	Mar 5, 1976	360	1.58	.00602
23	Feb 5, 1976	940	5.83	.00504

in regression analysis.

Impervious Percent	Maximum precipitation in inches over indicated time interval in minutes					
	5	15	30	60	120	180
32	.52	1.41	2.54	2.60	2.86	2.86
32	.46	.83	.97	.99	.99	.99
38	.28	.48	.67	.76	1.16	1.28
38	.15	.37	.53	.77	.93	1.06
38	.32	.75	.84	.84	1.11	1.28
5	.13	.31	.47	.67	1.13	1.20
5	.23	.55	.90	1.31	1.55	1.66
7	.20	.43	.49	.87	.93	.93
7	.40	.68	.86	1.00	1.17	1.18
7	.25	.48	.56	.67	.72	.73
2	.10	.18	.29	.45	.63	.83
2	.30	.59	.75	.88	1.09	1.19
2	.19	.36	.69	.94	1.07	1.13
27	.11	.22	.38	.54	.64	.66
27	.21	.38	.51	.67	1.27	1.55
27	.26	.61	1.07	1.43	1.64	1.64
21	.25	.41	.47	.65	.81	.86
21	.10	.17	.26	.45	.63	.86
21	.15	.24	.44	.84	1.01	1.01
21	.13	.22	.39	.50	.81	.85
21	.23	.55	.90	1.31	1.55	1.66
21	.25	.44	.54	.54	0.63	0.67
38	.16	.36	.54	.75	1.28	1.54
38	.12	.17	.20	.22	.22	.27
38	.36	.78	1.11	1.12	1.17	1.17
46	.28	.74	1.17	1.34	1.39	1.39
46	.12	.35	.57	.74	.98	1.30
54	.13	.33	.61	.82	.95	.96
54	.22	.33	.52	.73	1.02	1.03
54	.15	.30	.54	.72	1.01	1.31
54	.22	.35	.44	.74	.94	.94
59	.17	.37	.55	.83	1.49	2.00
59	.21	.47	.54	.60	.72	.84
59	.23	.44	.63	.88	.89	.89
59	.14	.29	.39	.61	.86	.86
54	.33	.78	1.00	1.09	1.16	1.28
46	.28	.51	.71	.82	1.05	1.14
46	.22	.39	.54	.79	1.20	2.06
46	.19	.29	.47	.67	.88	.89
46	.19	.39	.50	.50	.60	.61
2	.26	.39	.55	.79	1.20	1.21
2	.20	.49	.76	1.01	1.33	1.49
4	.17	.36	.46	.67	1.04	1.04

Table 3.--Data matrix used

Stream Site	Storm Date	Peak Discharge (ft^3/s)	Drainage Area (mi^2)	Stream Slope (ft/ft)
23	Feb 17, 1976	680	5.83	.00504
23	Mar 5, 1976	930	5.83	.00504
23	Mar 29, 1976	1060	5.83	.00504
24	Nov 30, 1975	2000	19.40	.00354
24	Jun 25, 1976	1290	19.40	.00354
25	Apr 8, 1975	270	0.59	.00511
25	May 11, 1975	200	0.59	.00511
25	Sep 9, 1975	220	0.59	.00511
28	Mar 12, 1975	1090	2.63	.00339
28	Mar 29, 1976	940	2.63	.00339
31	Mar 12, 1975	45	0.05	.01333
31	Mar 28, 1975	39	0.05	.01333
31	May 11, 1975	31	.05	.01333
31	Aug 27, 1975	38	.05	.01333
31	Nov 20, 1975	24	.05	.01333
31	Jan 27, 1976	34	.05	.01333

in regression analysis (continued).

Impervious Percent	Maximum precipitation in inches over indicated time interval in minutes					
	5	15	30	60	120	180
4	.22	.42	.59	.71	.96	1.18
4	.29	.54	.83	1.12	1.34	1.49
4	.36	.65	.87	.92	1.07	1.17
5	.16	.25	.36	.60	.92	1.26
5	.07	.15	.24	.46	.67	0.88
49	.16	.35	.59	.73	1.13	1.15
49	.17	.44	.61	.66	.79	.79
49	.19	.53	.54	.56	.72	.73
40	.47	.70	.91	1.15	1.31	1.33
40	.31	.40	.55	.56	1.09	1.44
69	.26	.55	.69	.76	1.07	1.25
69	.15	.30	.40	.51	.58	.59
69	.40	.78	.86	.89	1.00	1.02
69	.25	.70	.92	.99	1.01	1.01
69	.10	.22	.23	.26	.33	.43
69	.33	.81	.96	1.00	1.01	1.01

have relatively lower percentages of impervious area, and the rural areas have near zero impervious area. Some basins contain localized developments with impervious areas considerably different from the rest of the basin. In the eventual modeling scheme these areas will be treated individually, but for the present report only the average imperviousness of each entire basin is considered.

The impervious area parameter was determined using a fine grid overlay and aerial photography at 1:12,000 scale by counting the number of grid units that fell on impervious areas such as buildings, parking lots, and streets in the basin. The impervious percent, IMP, was then computed from the ratio of grid units on impervious area to total grid units in the basin. The technique is considered as accurate as any other readily available technique.

Other urbanization factors that probably affect runoff characteristics are the extent of development of storm sewers, the density of storm sewer inlets, the landscaping designs, and the modification of hillsides for developments. These factors will be evaluated for the ultimate modeling applications later in this project. They are, however, expected to be highly related to the imperviousness of the basins. For this preliminary analysis, the percentage of each basin that is impervious is the only urbanization characteristic that is considered.

Slope -

Basin and stream slope are important factors controlling the rate that storm-water will flow from the drainage area. The index of slope used in this study is the dimensionless factor of feet change in elevation per foot of channel length. This index is comparable to one developed by Benson (1962), who expressed slope in feet change in elevation per mile of channel length between the points 10 and 85 percent of the distance from the gaging site to the basin border. For the present report these slopes were computed by measuring distance and change in elevation on contour maps. For small basins, these slopes will be verified by field surveys later in the project work, but for this report the use of the maps is considered sufficiently accurate. The symbol S is used to indicate basin slope in this study.

THE ANALYTICAL TECHNIQUE

The procedure used in this report to develop a relation for expression of flood magnitude is called "multiple regression analysis." This technique has been used in many states, including Tennessee (May and others, 1970 and Wibben, 1976), to relate discharge of a given frequency to various channel and basin characteristics. In the present analysis, however, the discharges used are of various magnitudes of unknown frequencies, and each discharge is related not only to certain basin parameters but also to the rainfall that caused the flood event. The discharges used are those that occurred on different streams; these flows provide input to a single statistical model, rather than calibrating such a model or a parametric model for each stream. By this technique, a single relationship can be developed that will express, for any stream, the flood magnitude that will result from a given storm or storm characteristics.

The multiple-regression-analysis technique provides a mathematical relation between a dependent variable and various independent variables. In the present analysis, the dependent variable is peak discharge. The discharges used are flood discharges that have been observed at gaging stations and are limited to those for gaging stations that have reasonably well-defined stage-discharge relations. The selection of storms for analysis was also partially governed by the availability of reliable rainfall data. The measured rainfall must be representative of precipitation that fell over the entire basin. The reliability of questionable storm rainfall data was evaluated by considering the distribution of rainfall as defined by the entire gage network.

The independent variables that are used in the regression analysis include the channel and basin characteristics discussed earlier; drainage area (A), imperviousness (IMP), and slope (S). Also included as independent variables are the maximum rainfall amounts that occurred for each storm over 5-minute (I5), 15-minute (I15), 30-minute (I30), 60-minute (I60), 120-minute (I120) and 180-minute (I180) time intervals. These variables are given in table 3.

The data matrix of table 3 was not used directly in the multiple regression analysis. Previous studies (Thomas and Benson, 1970) have shown that streamflow characteristics are related linearly to most basin characteristics if the logarithms of each are used. Graphic plots of peak flow per square mile of basin versus rainfall magnitude for the 30-minute time interval also indicate the general applicability of a linear regression model for the logarithms of the variables. Accordingly, in conducting the analysis, all values for the streamflow, channel and rainfall characteristics were transformed to their logarithmic equivalents before calculations were attempted.

The independent variables, in most analyses, are not totally independent. A requisite, however, in regression analysis, is that the independent variables not be highly related amongst themselves. Violation of this criterion can lead to unstable values for the regression coefficients and to difficulties in evaluating the effectiveness of each independent variable. To show the degree of interdependence, a simple correlation matrix of the independent variables used in the analysis is given in table 4. In this table, a coefficient of 1.00 indicates perfect correlation, a value of 0 describes complete independence, and a value of -1.00 defines perfect inverse correlation.

The multiple regression technique provides, in addition to an equation, a measure of the accuracy of the defined relation and a measure of the significance of each independent variable. The process can be controlled to retain in the equation only those variables that have a selected level of effectiveness on the relationship. For this analysis only those independent variables that had a 95-percent probability of effectiveness were considered significant and were retained in the equation.

The initial analyses of the available data by linear multiple regression of the logarithms of the variables provided results that appeared reliable. The technique produced an equation that expressed flow as a function of drainage area, percent impervious, and two-hour rainfall. The standard error of estimate of the model was well within what was considered acceptable, however, some deficiencies in the model were apparent upon close examination and comparison of observed and simulated discharge magnitudes. This comparison indicated that, in a given basin, simulated discharges for the smaller storms were larger than the observed flows, and the simulated flows for the larger storms were less than the observed flows. This reflected a non-linearity of the logarithms of flow and rainfall. Such a non-linearity was not expected, however, it can be theorized to be caused by reduced infiltration of rainfall during the more severe storms as soil saturation occurred.

A second non-linearity, between logarithms of flow and drainage area, was anticipated as a result of the analyses by Hauth (1974) of flood frequency relations for small streams in Missouri. In the present study, this non-linearity became apparent when annual flows generated by the model for non-urbanized basins using long term Memphis rainfall were subjected to a frequency analysis. Relations between the logarithms of flood characteristics and the logarithms of drainage area were not the same slope as were similar relations developed by May and others (1970) or Randolph and Gamble (1976) for areas in West Tennessee. The data used in those analyses were from drainage areas generally much larger than the small streams in Memphis, and this

Table 4.--Correlation matrix of independent variables used in regression analysis.

	A	S	IMP	I5	I15	I30	I60	I120	I180
A	1.00	-0.85	-0.48	-0.01	-0.14	-0.13	-0.04	0.07	0.20
S		1.00	0.25	0.01	0.08	0.07	0.03	-0.06	-0.15
IMP			1.00	0.06	0.13	0.07	-0.07	-0.12	-0.18
I5				1.00	0.91	0.83	0.69	0.56	0.35
I15					1.00	0.94	0.79	0.63	0.42
I30						1.00	0.92	0.79	0.58
I60							1.00	0.91	0.74
I120								1.00	0.90
I180									1.00

change in slope of the relation with increasing drainage size indicated the non-linearity and possible discontinuity of the relations. A continuous relation should exist for all basin sizes for flows of a given frequency in a given area. Although the analysis of runoff in the Memphis area does not consider streams with large drainage basins, the developed relations should be compatible with established relations for the area. A curvilinear model similar to that suggested by Creager and others (1947) was adopted because of the deficiencies in the linear regression model. The use of Creager's model was found to be satisfactory by Hauth (1974) in developing flood-frequency relations for Missouri streams.

For the Memphis area, Creager's principle was further used to modify the linear regression model to account for the non-linearity in the relations of the logarithms of flow and two-hour rainfall. The resulting equation is of the form:

$$Q = b\,A^{a_1 A^{a_2}}\,(IMP + 1)^{a_3}\,(I120)^{a_4 (I120)^{a_5}}$$

where A, IMP and I120 are as previously defined, and

a_1 - a_5 and b are regression coefficients.

By logarithmic transformation the model becomes:

$$\log Q = \log b + a_1 A^{a_2} \log A + a_3 \log(IMP + 1) + a_4 (I120)^{a_5} \log(I120)$$

The exponents in this relation cannot be optimized by the linear multiple regression analysis to produce the optimum equation for simulating peak flow. However, trial and error solutions of the equation for varying values of a_2 and a_5 indicated that when $a_2 = -.02$, and $a_5 = 0.40$, the multiple regression analysis provided relations with minimum standard error of estimate.

ANALYTICAL RESULTS

The generalized equation developed by use of the described multiple regression analysis is:

$$Q=158.3A^{.777A^{-.02}}(IMP+1)^{.227}(I120)^{.539(I120)^{.40}}$$

In this equation, 158.3 is a regression constant, Q, A, IMP, and I120 are variables as previously defined and the exponents of the variables are the regression coefficients. The accuracy of this relation is indicated by a standard error of estimate (SE) of .083 logarithmic units. This SE implies that the equation produces values that are within 19 percent of the true values for about two-thirds of the storms used in the regression. This SE also implies that the calculated values are within 38 percent of the true values for about 95 percent of the storms. To graphically illustrate the goodness of fit of the simulated values, a plot of the logarithms of observed versus calculated peaks is shown in figure 2. The capability of the relation to simulate storm flow is also indicated by a correlation coefficient of 0.98.

The defined relation includes all of the basin characteristics that were considered in the analysis except basin slope. Slope (S) has a high inverse correlation with drainage area, and probably for this reason the influence of S on the flood peak is minimal. The precipitation variable retained by the analysis is the rainfall amount for the 120 minute interval. The rainfall variables have, as given in Table 4, a high degree of interdependence or correlation. I120 was found to be the most significant rainfall variable, however, the use of I180, or I60, produced equations only slightly weaker than the presented result.

The sensitivity of the defined relation to variations in the values of the independent variables is shown in figures 3-8. Figures 3-5 define actual change in discharge as the value of a single independent variable is changed; figures 6-8 show the errors that result from using erroneous values of the independent variables.

The highly significant effect of drainage area on discharge shown in figure 3 was expected. The curvature of the logarithmic plot illustrates the effect of the $A^{-.02}$ exponent in the drainage area coefficient.

The effect of impervious area on flood discharge as shown in figure 4 is considered reasonable, but is considerably less than defined for the Houston, Texas area (Johnson and Sayre, 1973). In the Memphis area, the defined relation shows that increasing the impervious percentage of the basin from near zero to 25 per-

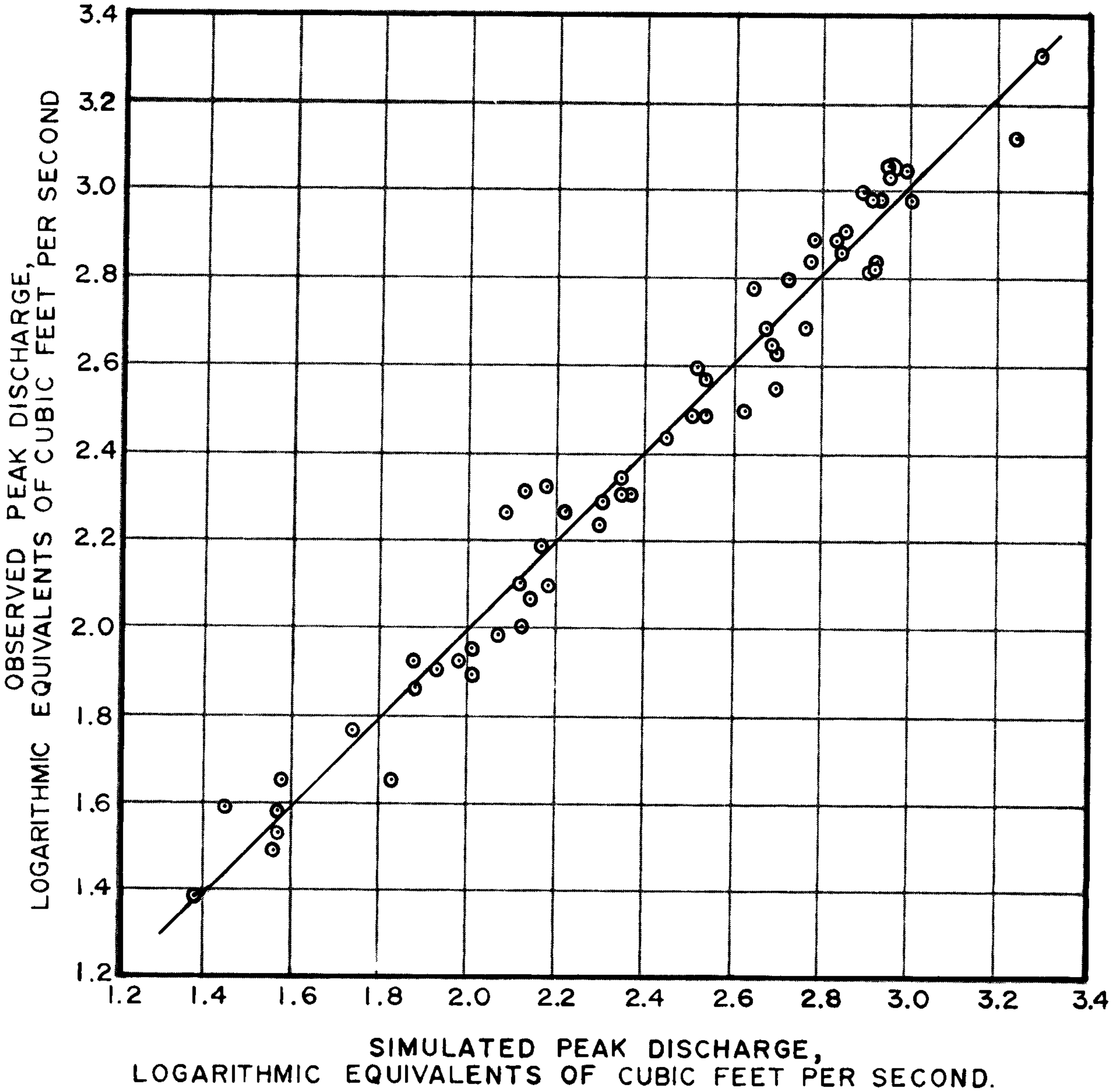


Figure 2.--Observed versus simulated peak flows used in calibration of the regression model.

cent causes the flood discharge to approximately double for a given storm. The relative effect of increased impervious area on flood runoff may be tempered by the soil type characteristic of the Memphis vicinity. This soil type, although exhibiting a large capacity for moisture, has a slow infiltration rate. Also the Memphis area basins are steeper than those in the Houston area, the channels draining non-urbanized areas are incised and well defined, and good storm runoff capability exists. This further reduces the relative effect of urbanization on storm runoff.

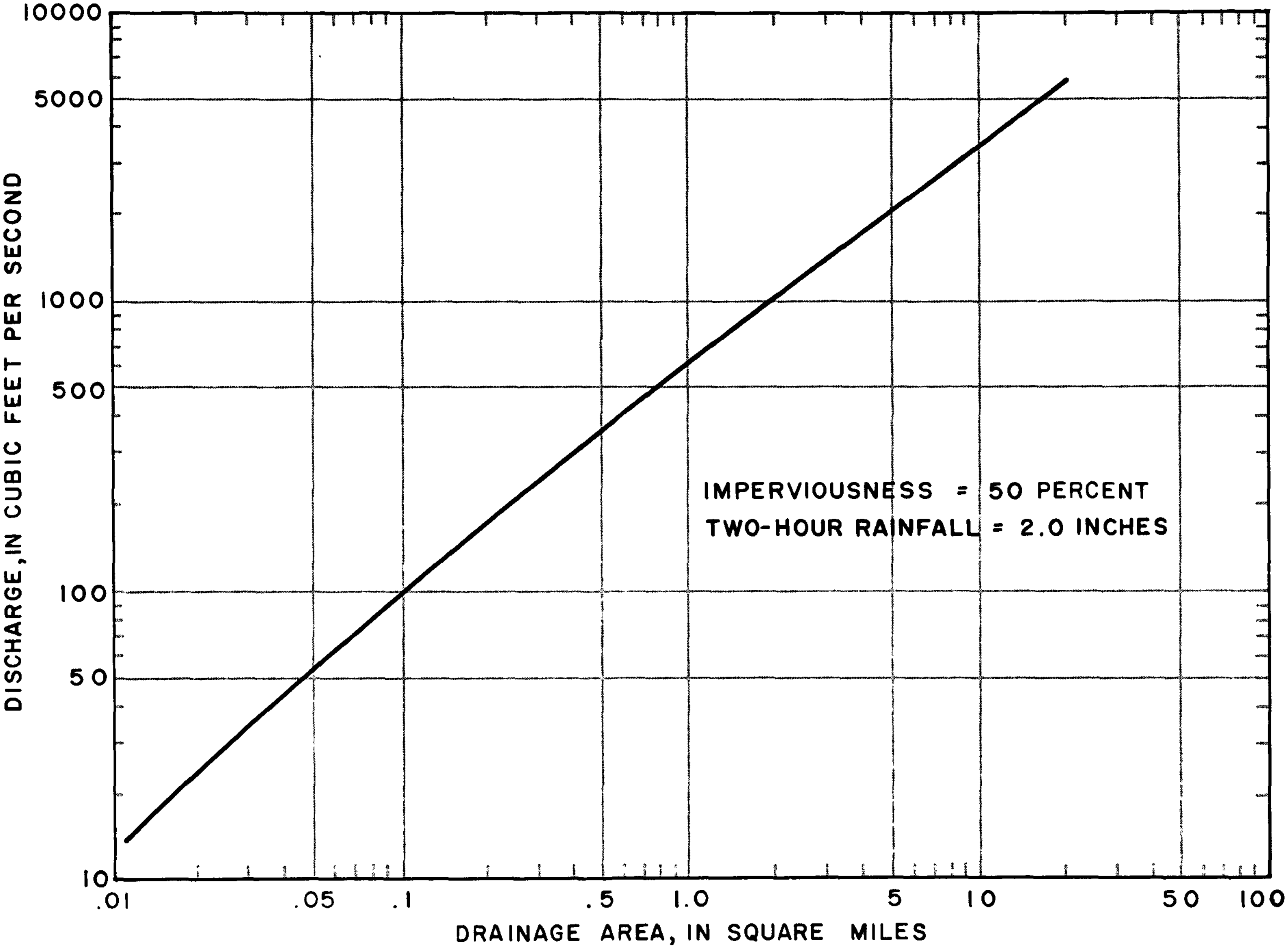


Figure 3.--Variation of discharge with variation in drainage area.

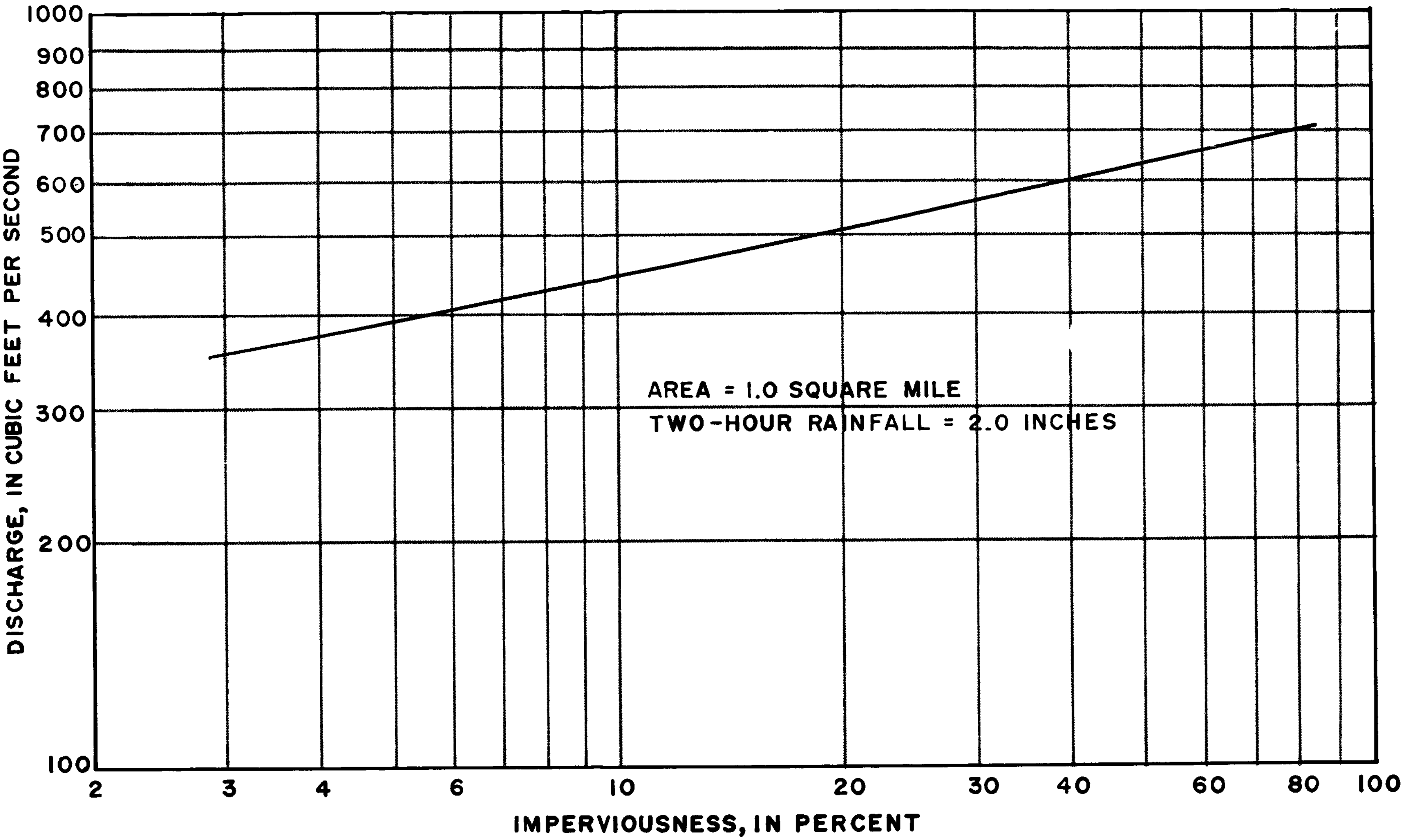


Figure 4.--Variation of discharge with variation in imperviousness.

The plot in figure 5, showing the variation of discharge with variation of two-hour rainfall, illustrates the non-linearity of the logarithms of these two variables. The effect of rainfall on discharge is very significant; a storm with a maximum two-hour rainfall of 3.0 inches will produce twice the peak runoff as a storm with a two-hour rainfall of 1.3 inches.

Drainage area is probably the most accurately determined independent variable used in the analysis. It is unlikely that computational errors in drainage area, even for extremely small basins, will exceed a few percent. Although discharge errors resulting from a given error in drainage area determination will vary with the size of the basin, this variation is not great as shown by figure 6. The error in predicted discharge caused by a plus 10 percent in computed area is about 8 percent.

Of the independent variables used in the defined relation, impervious area percentage may be the most difficult to accurately determine. However, figure 7 shows that a plus 50 percent error in percent impervious area for a basin, produced a discharge error of only about 10 percent. A minus 50 percent error in percent impervious area produced a discharge error of about 14 percent.

Accurate values of rainfall are relatively easily measured, however they may not be representative of precipitation throughout a stream basin. The storm may have a non-uniform areal distribution of rainfall; the rain-gage site may experience local wind effects causing the measured rainfall to vary from the average rainfall in the area. As shown in figure 8, errors in discharge caused by errors in measured two-hour rainfall vary with the storm intensity. A plus 50 percent in measurement of a true two-hour maximum rainfall of one inch, produces a discharge error of about 28 percent. A similar error in measurement of a two-hour precipitation of two inches, produces a discharge error of about 53 percent.

The reliability of the defined relationship is not fully indicated by the standard error of estimate or by the sensitivity analyses. The SE is a measure of the deviation of the input values from the line of regression, or in more general terms, a measure of the scatter of the points as illustrated in figure 2. A more reliable indication of the applicability of a regression equation can be determined through the use of split sample tests. Such tests evaluate the predictive reliability of the relations. Subsequent to selection of the initial set of data that was used to calibrate the regression model, additional storms and discharges became available. These 72 storms were used by the model to compute flood peaks, and a comparison of these predicted discharges to the observed discharges is shown in figure 9. These data were not screened for reliability as carefully as were the

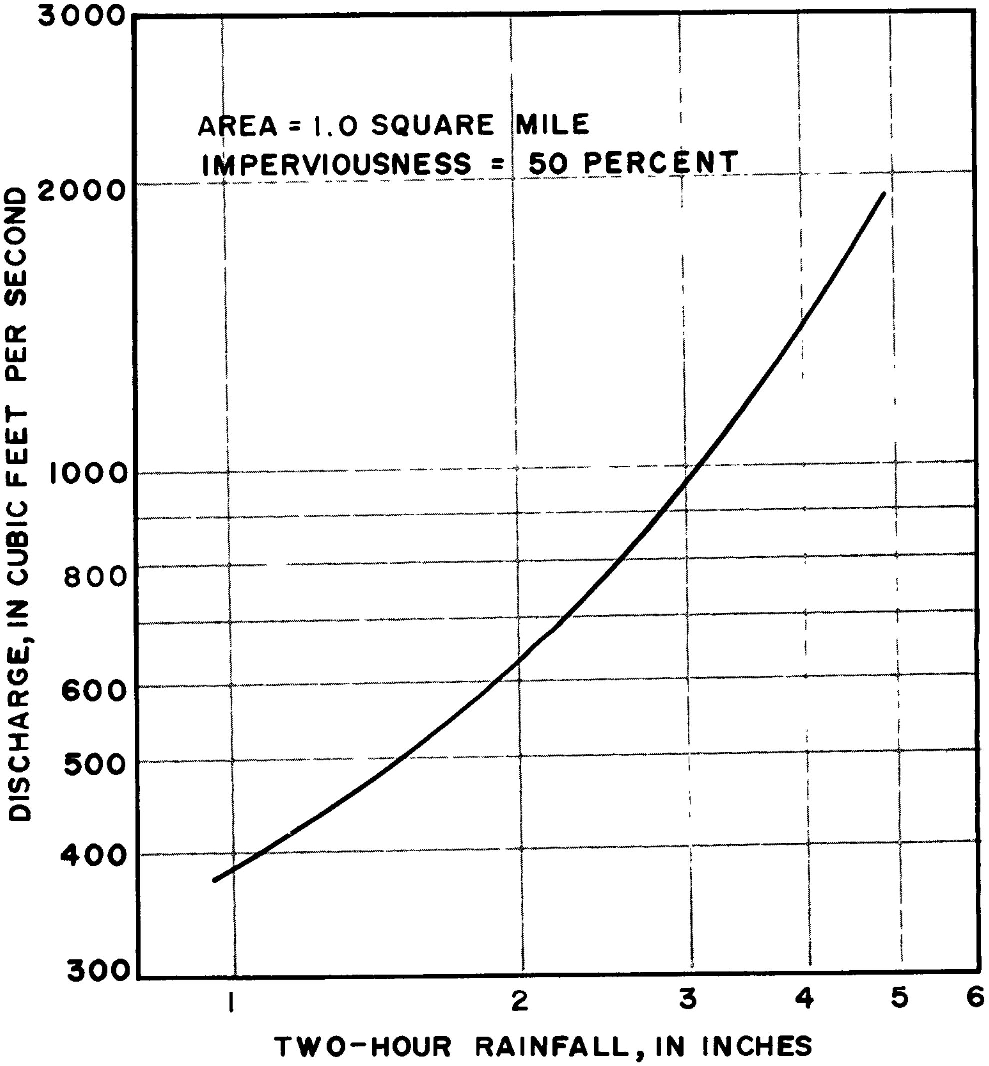


Figure 5.--Variation of discharge with variation in two-hour rainfall.

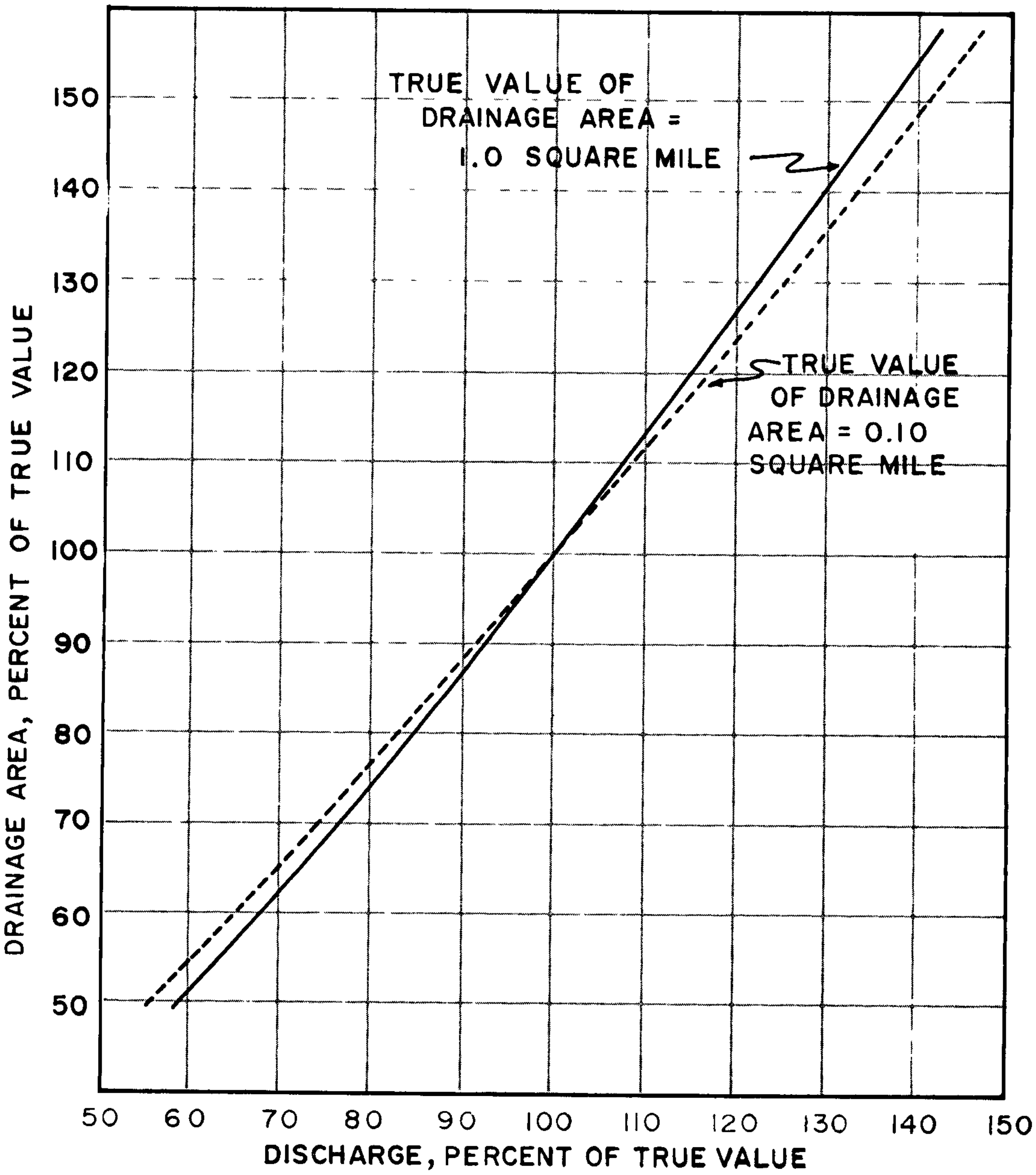


Figure 6.--Relation of discharge error to drainage area error.

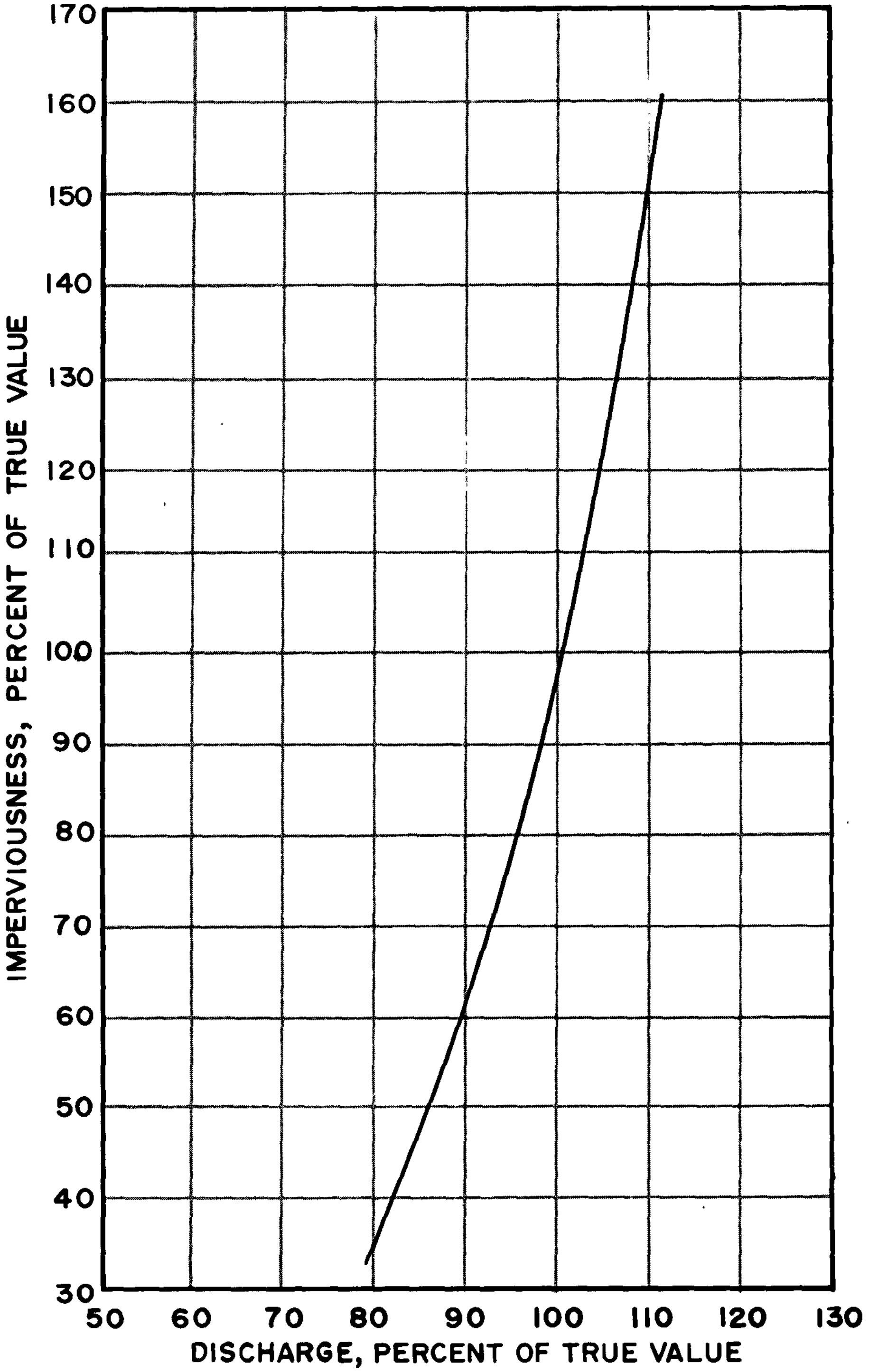


Figure 7.--Relation of discharge error to impervious percent error.

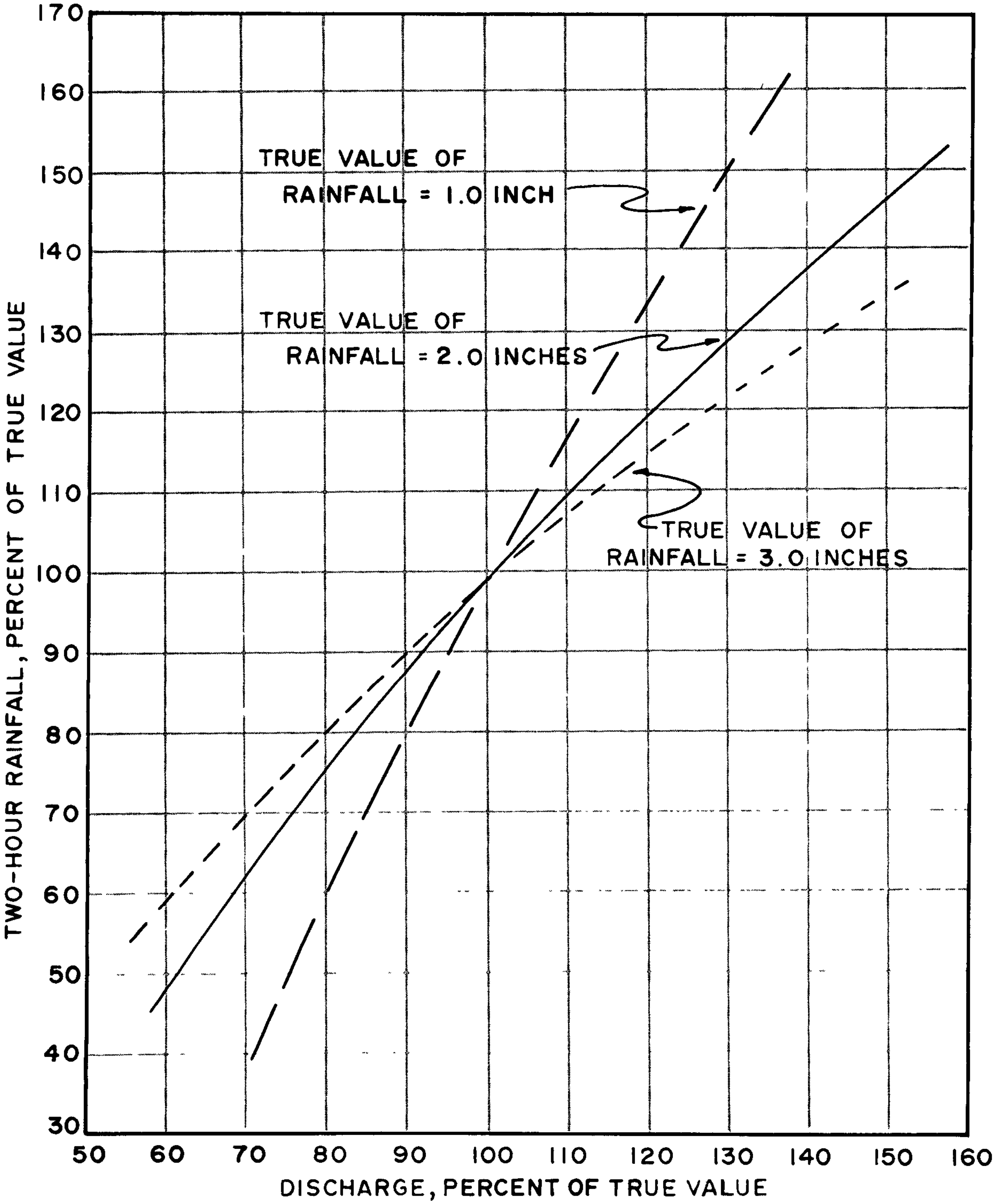


Figure 8.--Relation of errors in discharge to errors in 2-hour rainfall.

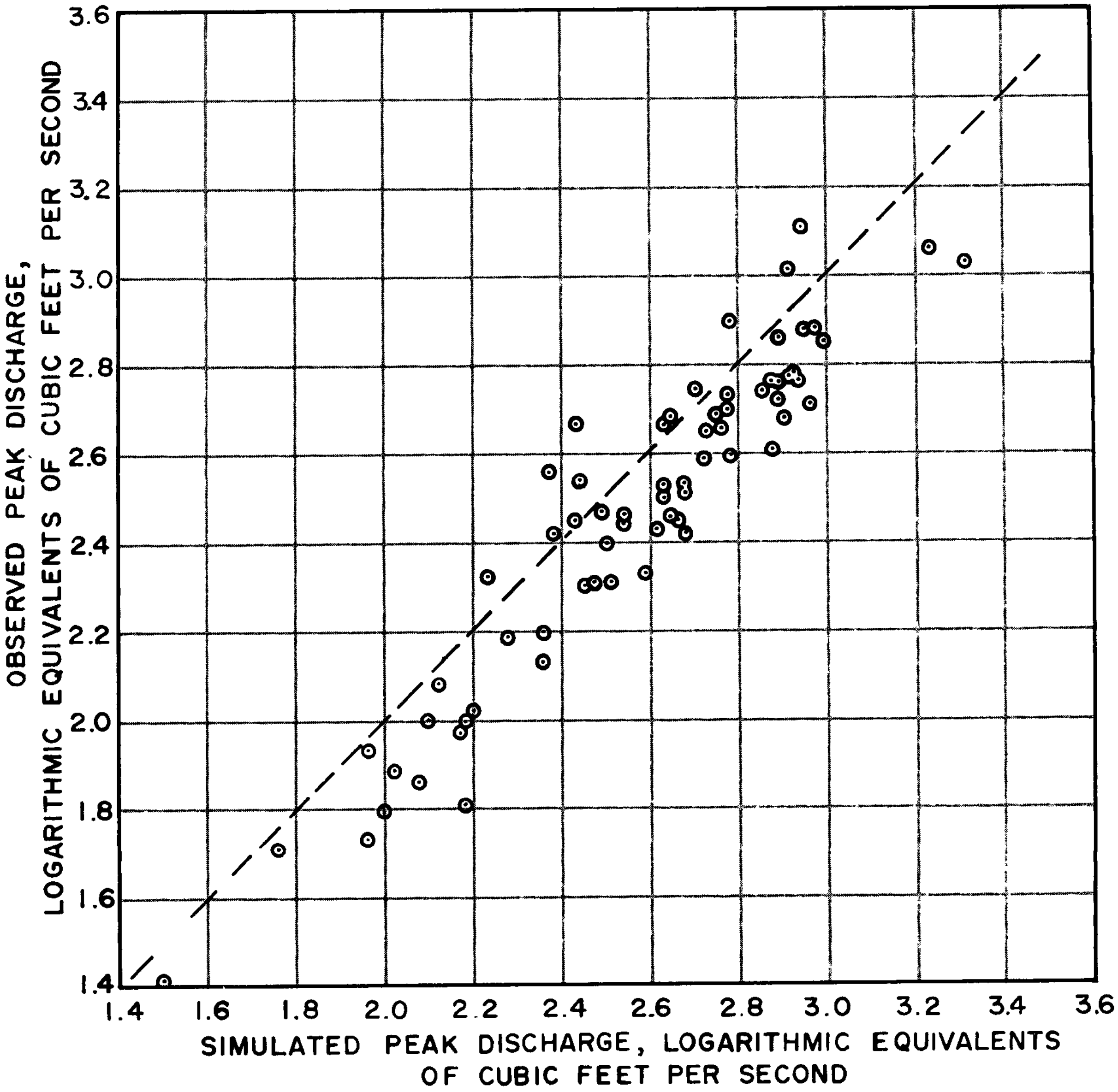


Figure 9.--Observed versus simulated peak flows used in test of regression model

calibration data and the comparison does show a bias in the prediction; the simulated discharges average about .05 logarithmic units greater than the observed discharges. The closeness of fit, however, and the standard error of prediction of .14 logarithmic or 27 percent, implies that the relation can be used with confidence to predict floods on small streams in the Memphis area.

APPLICATION OF THE GENERALIZED RELATION

The regression equation that was developed from observed peak discharges, associated storm rainfall, and basin characteristics, can be used with historical rainfall to synthesize peak flows on streams draining less than ten square miles in the Memphis area. These values can be further analyzed to determine flood magnitude and frequency relationships.

Significant storm data that occurred at Memphis from 1900 to 1976 were provided by the National Weather Service and the National Climatic Records Center. From these data, the annual maximum two-hour rainfall values were determined, and these are given in Table 5. The two-hour rainfall values were then used with the defined relations, to compute annual peak discharges for drainage basins of varying size and for varying degrees of imperviousness. Each resulting series of 76 annual flood peaks was then subjected to a log Pearson type III flood frequency analysis. Abbreviated results of these analyses are presented in figures 10 through 18.

The log Pearson type III distribution has been recommended by the Water Resources Council (March 1976) as the base method for flood frequency studies. A common criticism of this method is that it produces a distribution defining less flow at the 50 and 100 year frequencies than is generally defined by the previously and commonly used formula $T = \frac{n+1}{M}$ where T is recurrence interval, n is years of record, and M is the rank of the individual storm in the flood series. The use of this formula, in defining flood frequency, is depicted by the dashed lines in the plots in figures 10 through 18 for rural or zero percent impervious basins and for basins having 80 percent imperviousness. Curves for basins with other percentages of imperviousness have similar shapes. The 100 year flood peaks, defined by this formula and a graphical analysis, are about 25 percent greater than those defined by the log Pearson type III distribution. The graphical and tabular relations presented elsewhere in this report, however, are those defined by the log Pearson type III analysis.

Results from use of the log Pearson type III frequency analysis to relate discharge of various frequencies to drainage areas and degree of imperviousness in the Memphis area are shown by the following equations. The subscripts on the discharge symbol Q represents the frequency or recurrence interval of flow, the other symbols in the equation are as previously defined.

Table 5.--Annual maximum two-hour rainfall at Memphis.

Water Year (Oct. to Sept.)	Date	Two-Hour Rainfall (Inches)	Water Year	Date	Two-Hour Rainfall (Inches)
1901	Sep 14, 1901	1.90	1939	May 21, 1939	2.00
1902	Aug 1, 1902	2.10	1940	Jul 15, 1940	1.98
1903	Dec 15, 1902	1.45	1941	Jul 26, 1941	1.35
1904	Apr 24, 1904	1.35	1942	Oct 17, 1941	1.70
1905	Aug 10, 1905	2.25	1943	Sep 5, 1942	1.55
1906	Jul 16, 1906	1.90	1944	Apr 11, 1944	1.37
1907	Oct 19, 1906	1.55	1945	Sep 30, 1945	1.55
1908	Jul 24, 1908	1.39	1946	Jan 8, 1946	1.60
1909	Apr 6, 1909	1.65	1947	Jun 22, 1947	1.70
1910	Apr 14, 1910	1.27	1948	Oct 27, 1947	2.05
1911	Oct 5, 1910	1.50	1949	Apr 12, 1949	2.30
1912	Jul 16, 1912	1.99	1950	Jul 4, 1950	2.57
1913	Mar 20, 1913	1.63	1951	Jan 2, 1951	1.80
1914	May 4, 1914	2.08	1952	Aug 5, 1952	1.45
1915	Aug 19, 1915	1.95	1953	Jul 21, 1953	1.95
1916	Oct 18, 1915	1.30	1954	Nov 21, 1953	2.00
1917	Jul 19, 1917	1.51	1955	May 27, 1955	2.97
1918	Jun 5, 1918	2.45	1956	Aug 31, 1956	1.45
1919	Mar 16, 1919	2.22	1957	May 14, 1957	2.20
1920	Sep 7, 1920	2.85	1958	May 8, 1958	2.00
1921	Apr 15, 1921	1.19	1959	Jul 24, 1959	1.80
1922	Mar 18, 1922	1.47	1960	Aug 20, 1960	2.29
1923	Aug 29, 1923	1.90	1961	Apr 15, 1961	1.05
1924	May 20, 1924	2.46	1962	Aug 30, 1962	2.20
1925	Aug 9, 1925	1.45	1963	Apr 28, 1963	1.45
1926	Oct 1, 1925	2.30	1964	Apr 23, 1964	1.60
1927	Apr 20, 1927	2.05	1965	May 27, 1965	1.90
1928	Nov 8, 1927	1.70	1966	Jul 29, 1966	1.60
1929	Jul 16, 1929	4.70	1967	Aug 26, 1967	2.10
1930	May 18, 1930	1.38	1968	Dec 2, 1967	2.55
1931	Aug 19, 1931	0.76	1969	Apr 9, 1969	1.75
1932	Dec 30, 1931	1.45	1970	Jun 24, 1970	2.20
1933	Dec 30, 1932	1.60	1971	Feb 21, 1971	1.70
1934	Oct 15, 1933	1.19	1972	Jun 25, 1972	1.60
1935	Nov 21, 1934	3.53	1973	Apr 19, 1973	1.62
1936	Oct 22, 1935	3.48	1974	Jul 11, 1974	2.40
1937	Jul 4, 1937	2.03	1975	May 11, 1975	2.05
1938	Feb 17, 1938	1.40	1976	Sep 4, 1976	1.25

$$Q_2 = 228.6 \; A^{.777 A^{-.02}} \; (IMP + 1)^{.227}$$

$$Q_5 = 291.5 \; A^{.777 A^{-.02}} \; (IMP + 1)^{.227}$$

$$Q_{10} = 344.2 \; A^{.777 A^{-.02}} \; (IMP + 1)^{.227}$$

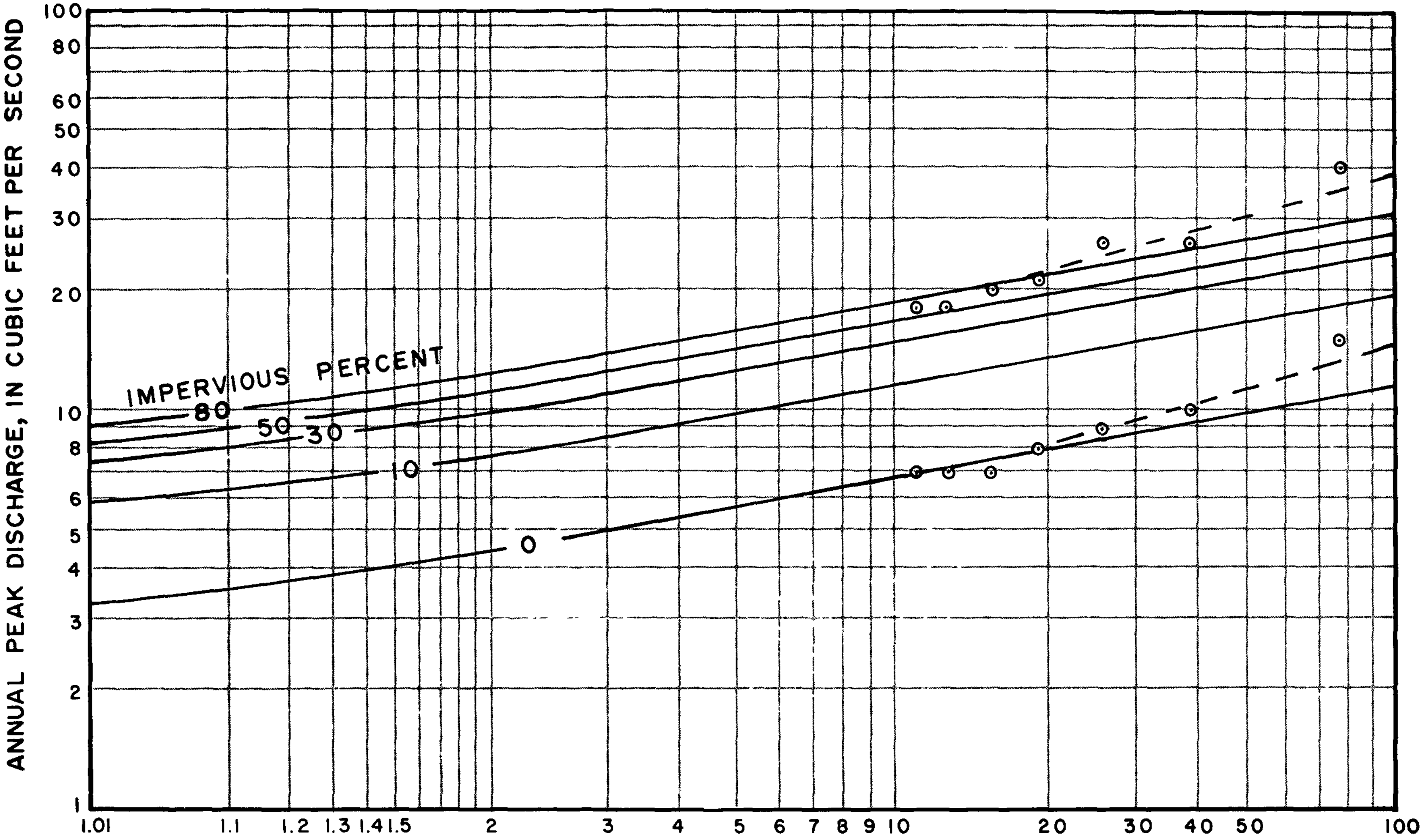
$$Q_{25} = 424.2 \; A^{.777 A^{-.02}} \; (IMP + 1)^{.227}$$

$$Q_{50} = 494.3 \; A^{.777 A^{-.02}} \; (IMP + 1)^{.227}$$

$$Q_{100} = 574.4 \; A^{.777 A^{-.02}} \; (IMP + 1)^{.227}$$

These relations for rural basins with zero percent imperviousness and for basins with 30 percent imperviousness are depicted in graphical form in figure 19. The shape of these relations are similar to the curve presented in figure 3. Figure 19, however, also shows the increase in flood flow caused by urbanization. To evaluate the "fit" of the developed relations, the results of analyses by Randolph and Gamble, (1976) and Hauth (1974) are also shown in this illustration. Hauth's equations include terms for basin slope. His curves, presented here, use slopes that vary with basin size as defined by the stream characteristics in the gaging station network in Memphis. The curves for rural conditions developed in this report for the Memphis area are quite close to those by Hauth, although the range in magnitude of the 2-year and 100-year flows is somewhat less than that for Missouri streams. For basins with drainages less than 10 square miles, there is an increasing disparity between the relations developed in this report and those presented by Randolph and Gamble. Although Randolph and Gamble's equations are based on basins as small as 0.29 square miles, many of the small streams records are only 11 years in length. Nevertheless, Randolph and Gamble's relations and the relations developed for rural streams in the Memphis area, do tend to merge for streams having drainage areas of about 10 square miles.

To illustrate the usefulness of the regression equation and the resulting flood frequency data that can be derived from its application with historical rainfall, the simulated peaks of various frequencies for hypothetical streams with various degrees of basin imperviousness were expressed in flow per acre and then plotted against drainage area. These data shown in figures 20-25 further indicate the relationships between peak runoff, drainage area, and imperviousness. Perhaps more significant, however, is the magnitude of floods in the range of frequencies (10 years or greater) generally used in storm drainage design. This analysis indicates that 2(ft^3/s)/acre that generally has been used for design purposes for many years in the Memphis area is inadequate in the urban environment for small basins. For large basins, the use of 2(ft^3/s)/acre provides an over design for storm drainage.



RECURRENCE INTERVAL, IN YEARS

Figure 10.--Flood - frequency relations for .01 square mile basin and 0 to 80 percent imperviousness.

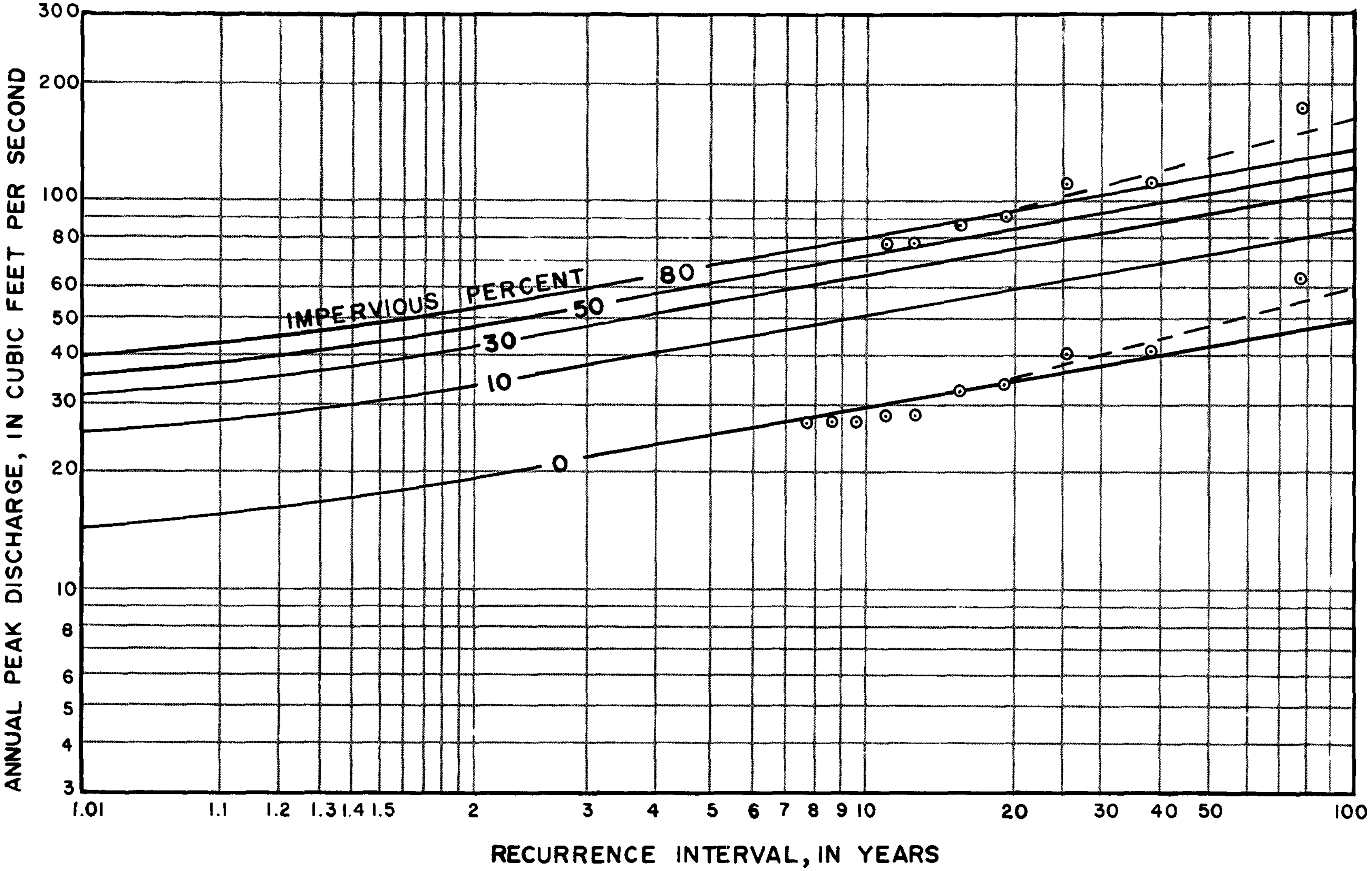


Figure 11.--Flood-frequency relations for .05 square mile basin and 0 to 80 percent imperviousness.

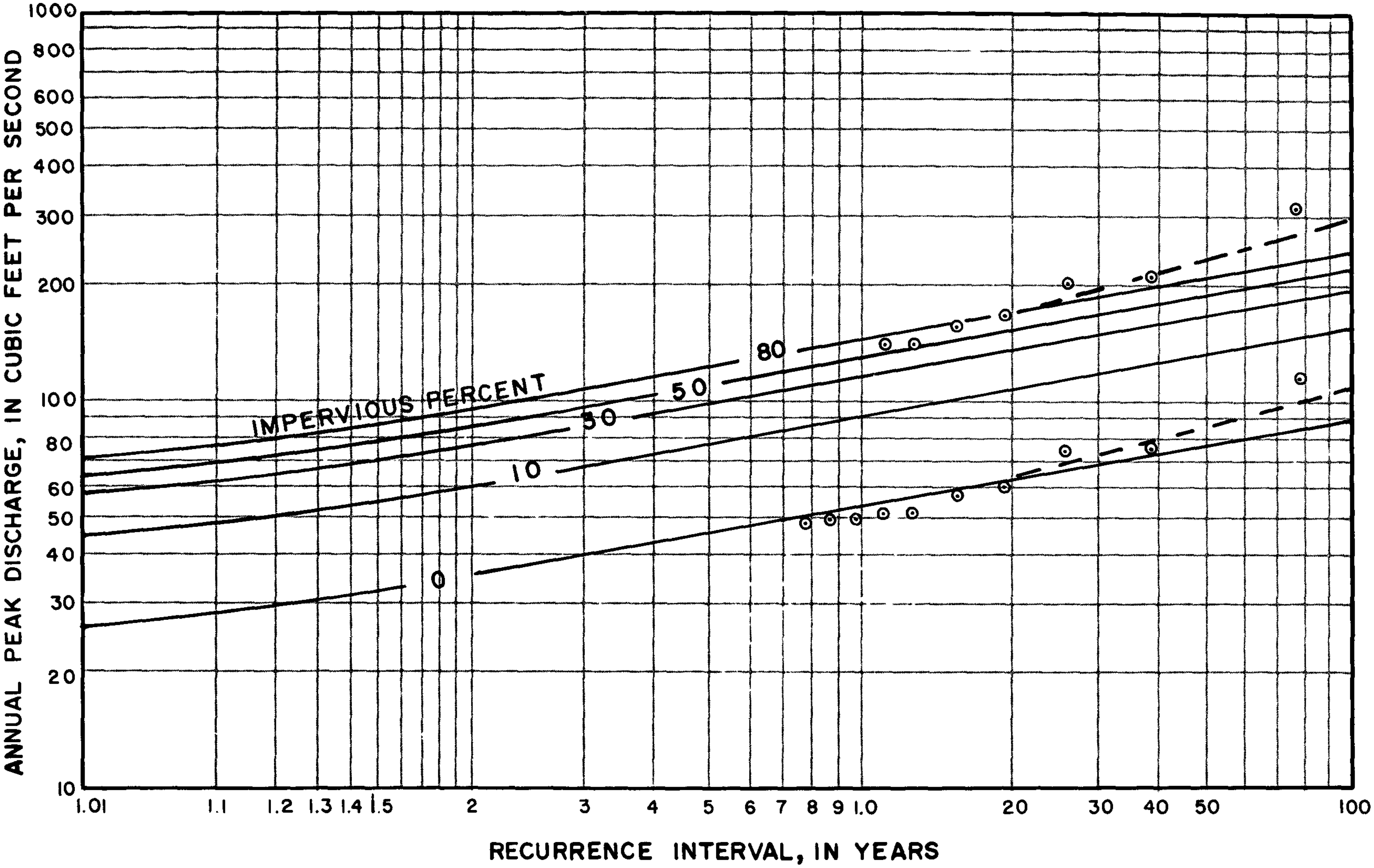


Figure 12.--Flood - frequency relations for .10 square mile basin and 0 to 80 percent imperviousness.

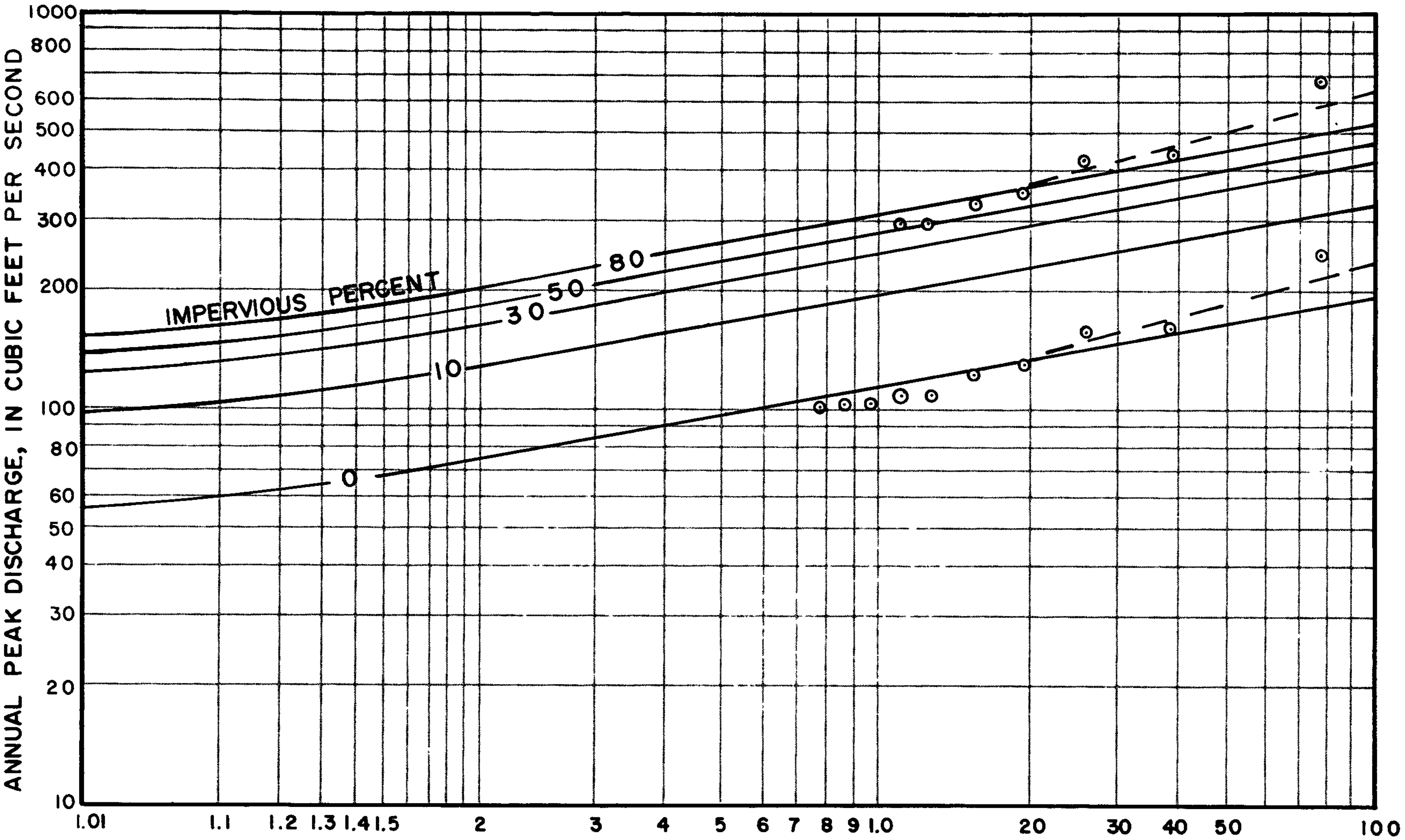


Figure 13.--Flood - frequency relations for 0.25 square mile basin and 0 to 80 percent imperviousness.

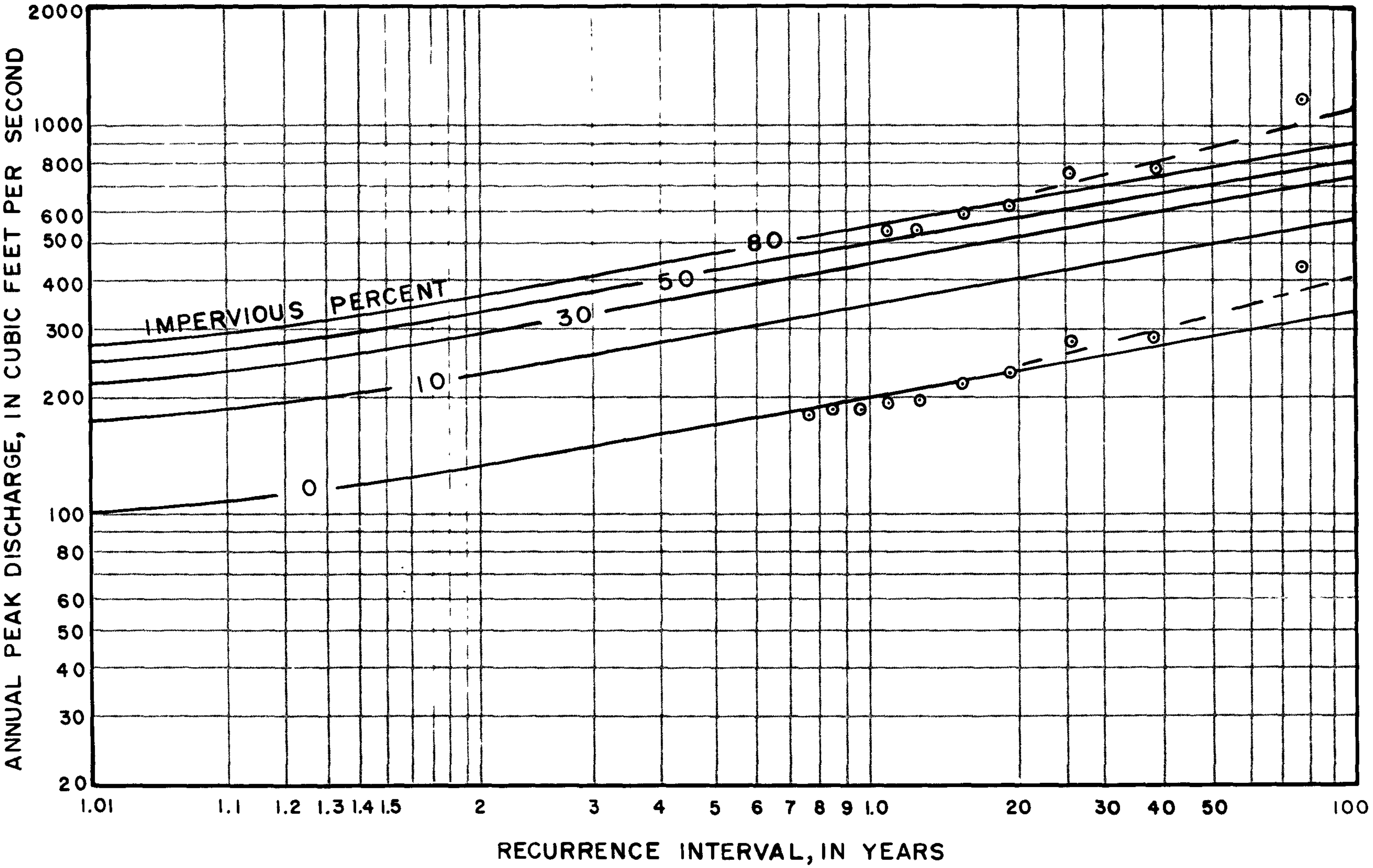


Figure 14--Flood-frequency relations for 0.50 square mile basin and 0 to 80 percent imperviousness.

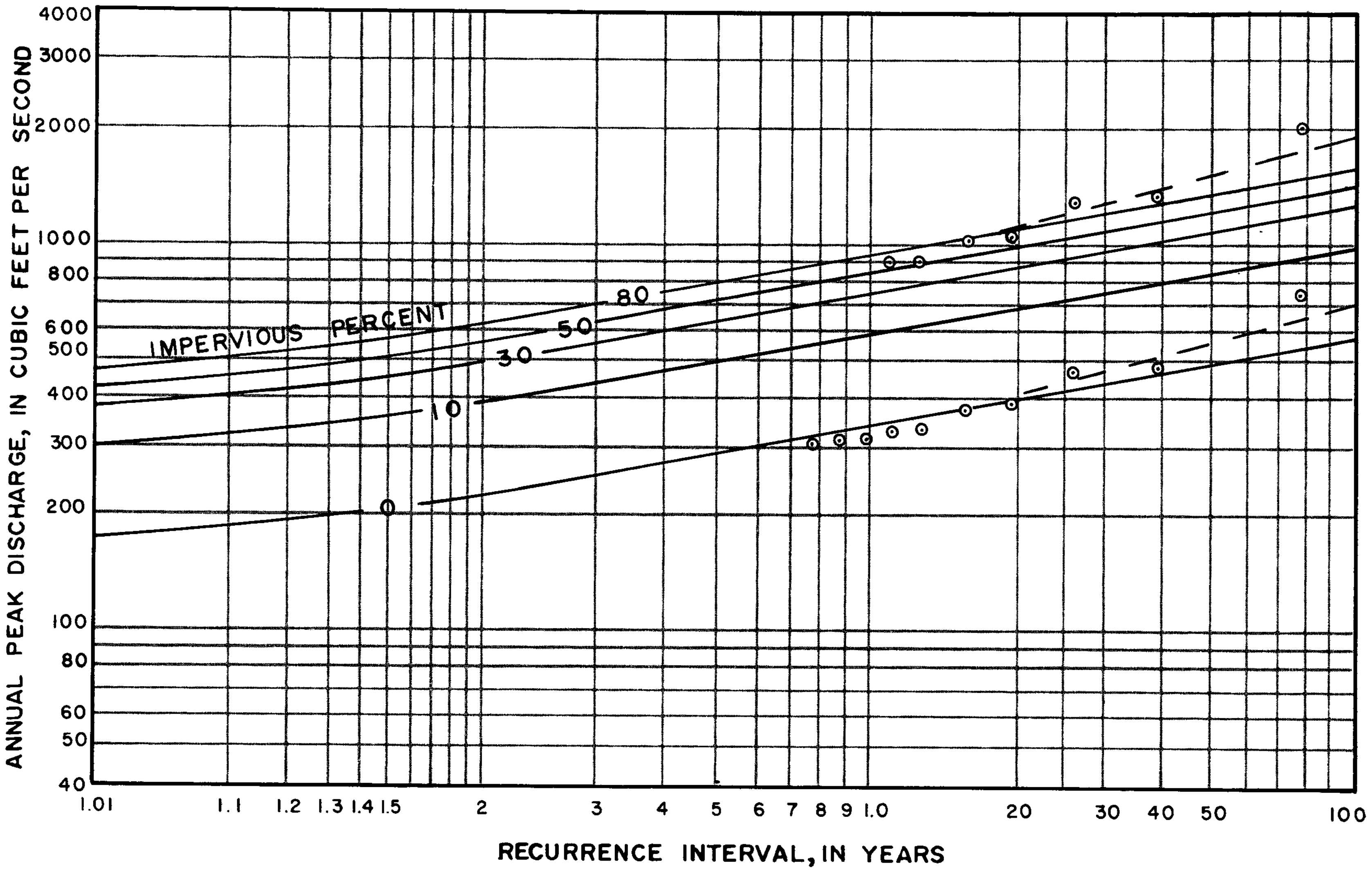


Figure 15.--Flood - frequency relations for 1.0 square mile basin and 0 to 80 percent imperviousness.

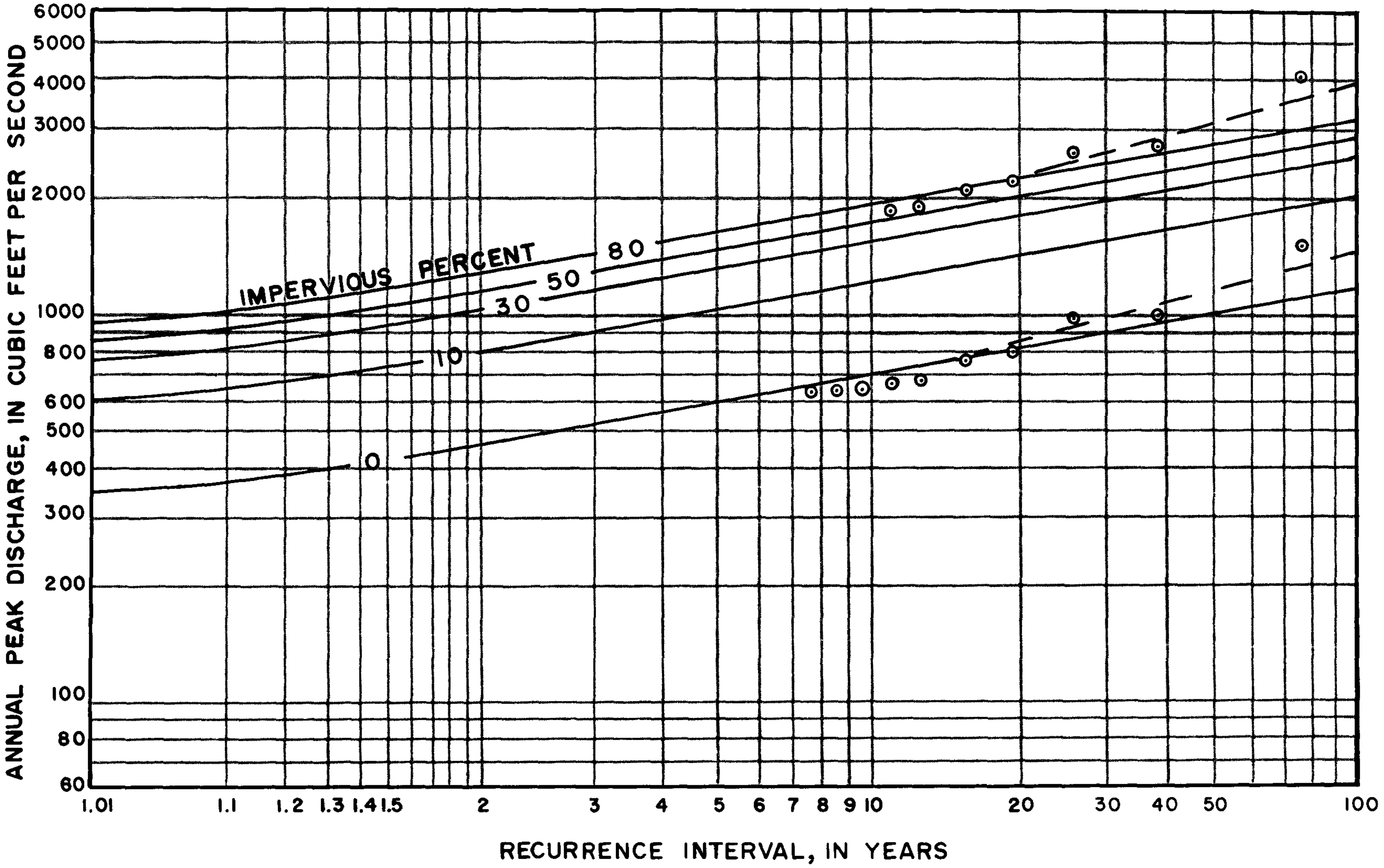


Figure 16.--Flood - frequency relations for 2.5 square mile basin and 0 to 80 percent imperviousness.

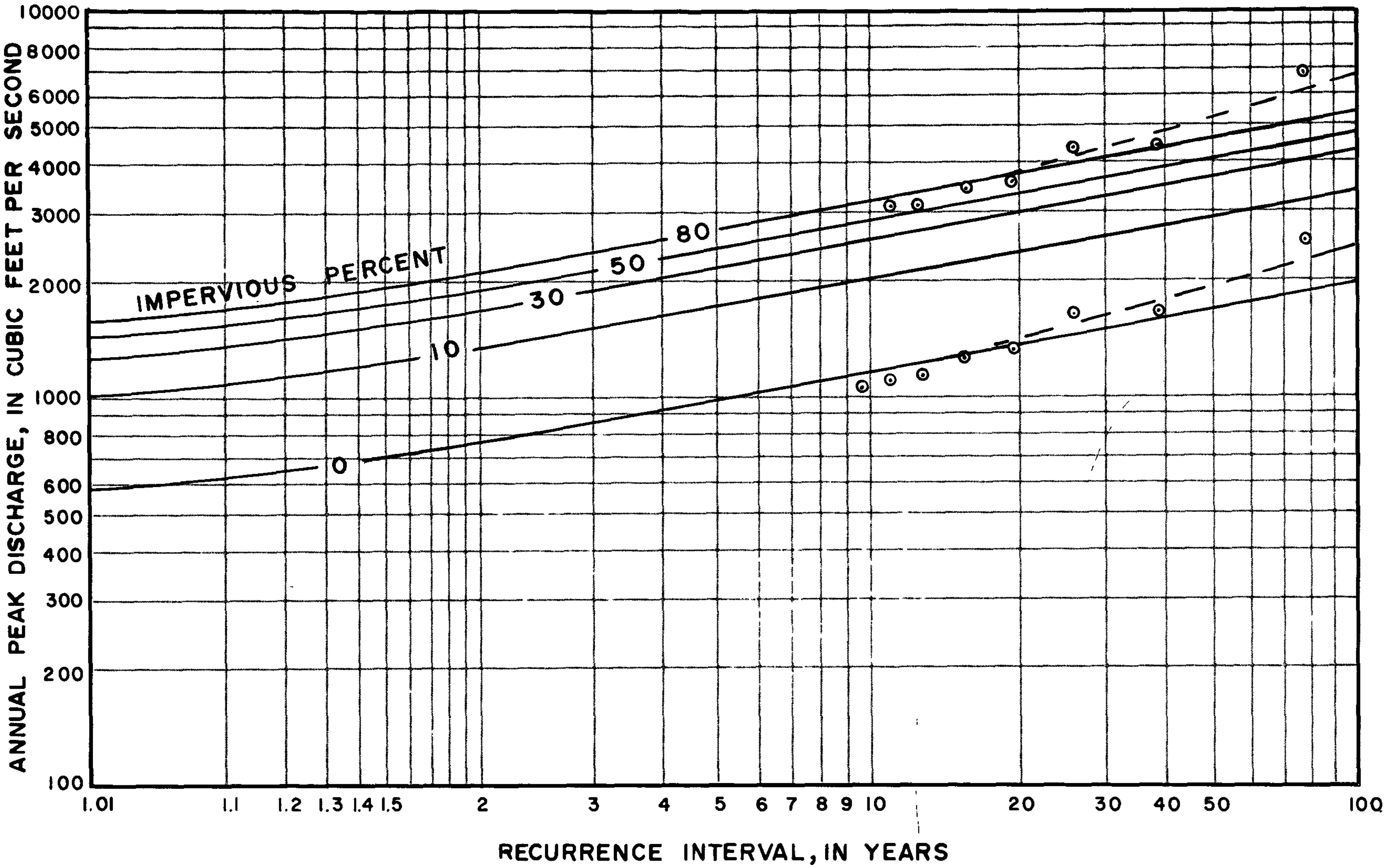


Figure 17.--Flood - frequency relations for 5.0 square mile basin and 0 to 80 percent imperviousness.

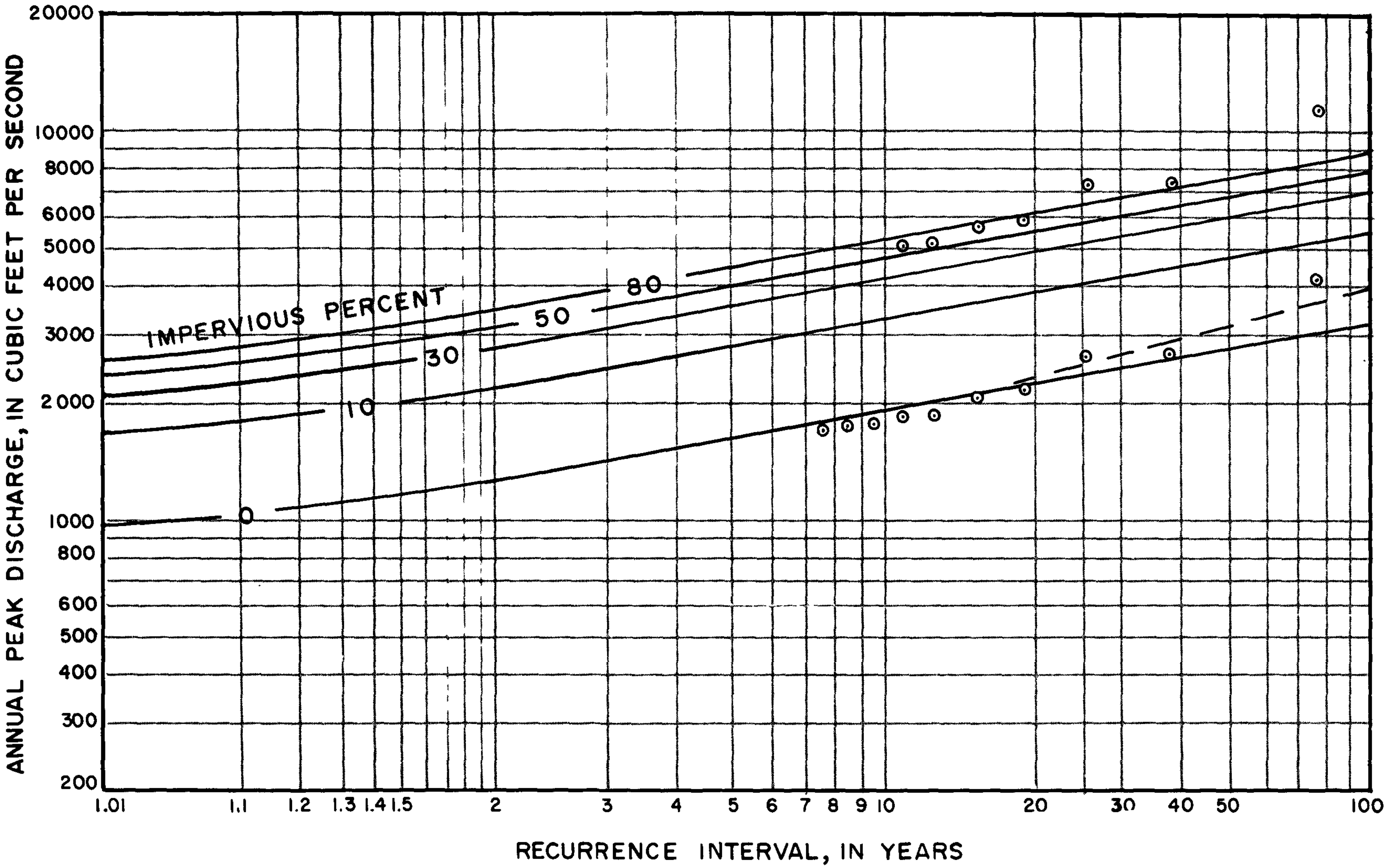


Figure 18.--Flood - frequency relations for 10.0 square mile basin and 0 to 80 percent imperviousness.

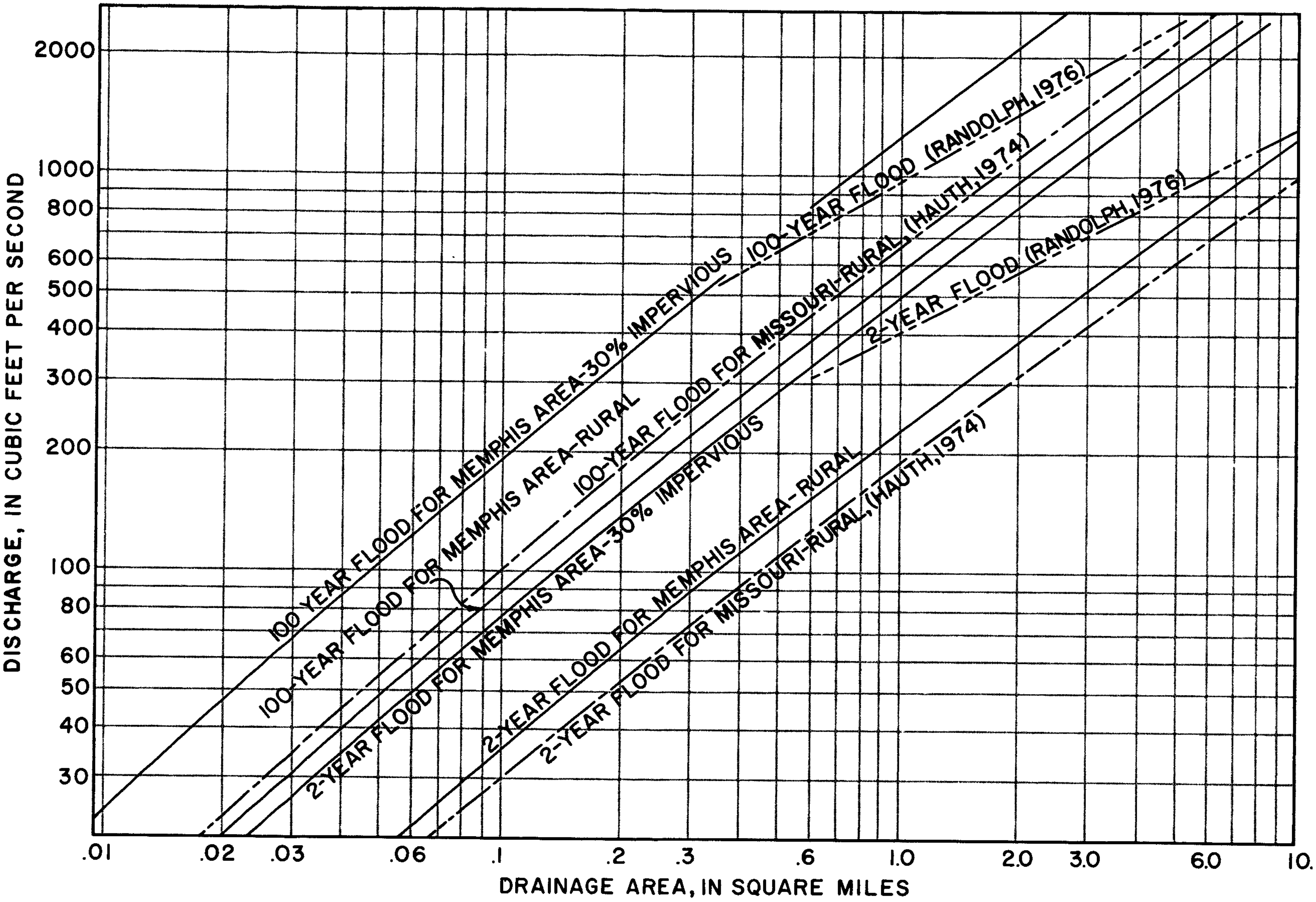


Figure 19.--Flood-frequency relations for rural conditions and for urban conditions with 30 percent imperviousness.

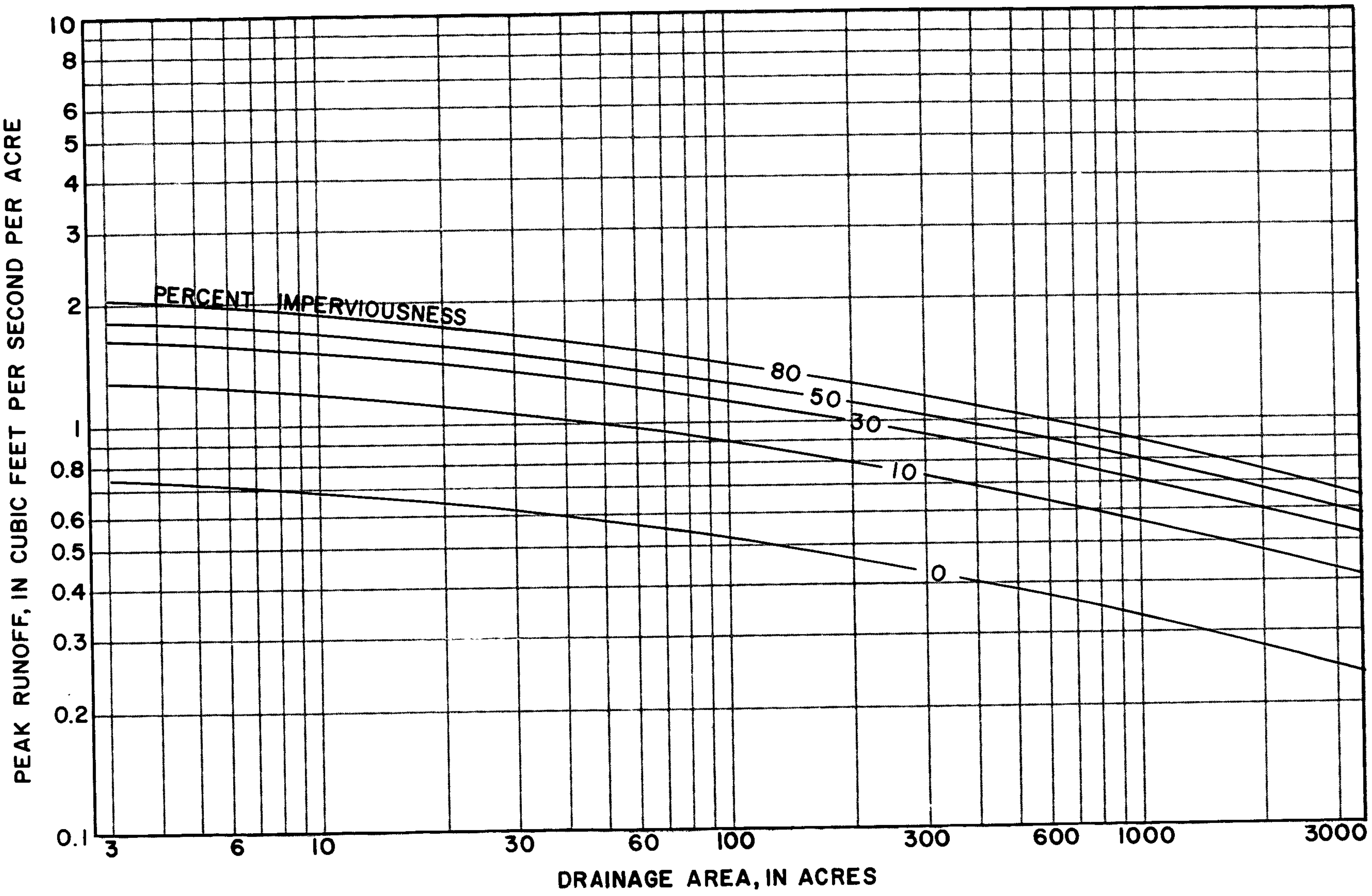


Figure 20.--Runoff per acre for 2-year floods.

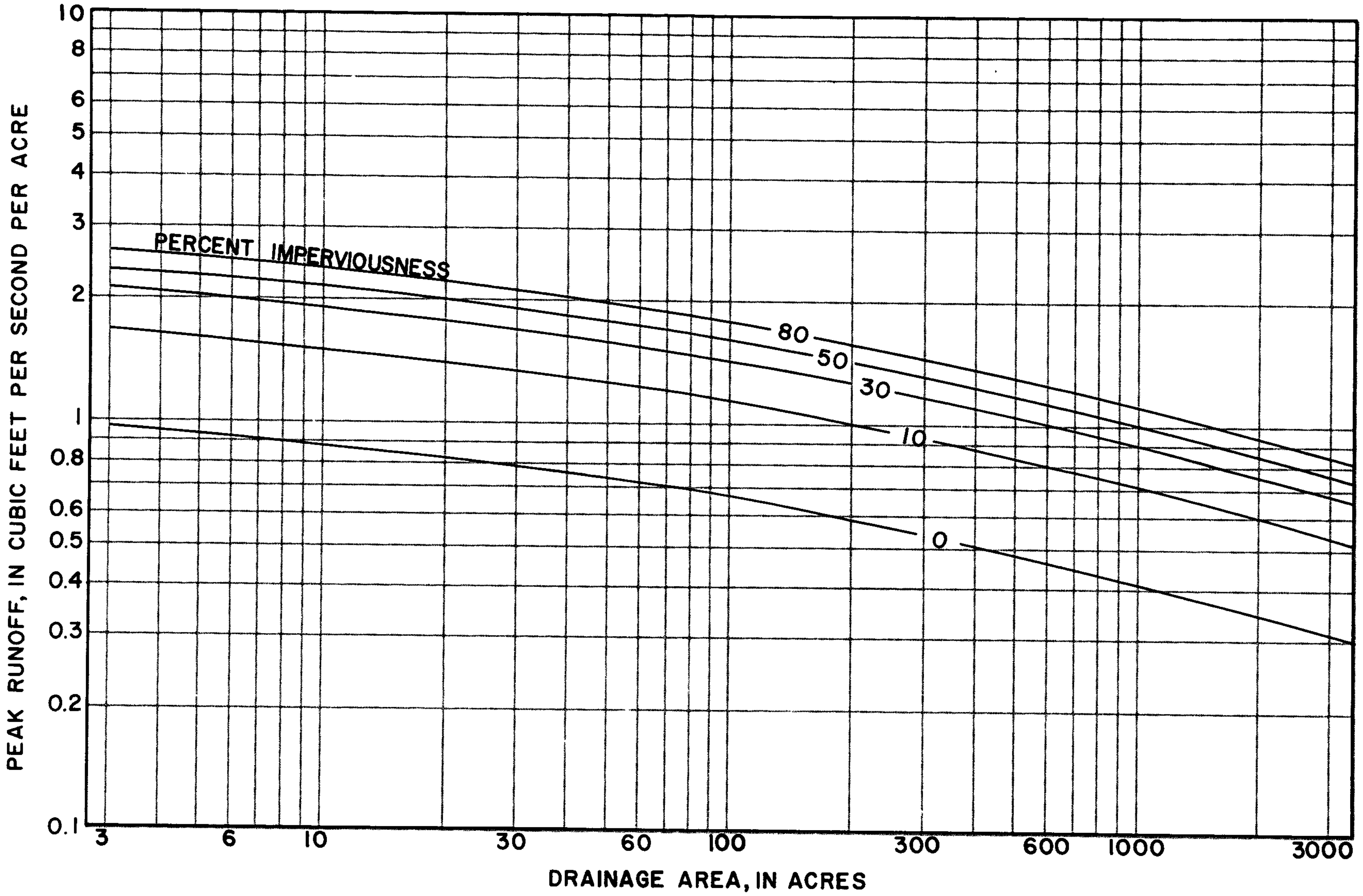


Figure 21.--Runoff per acre for 5-year floods.

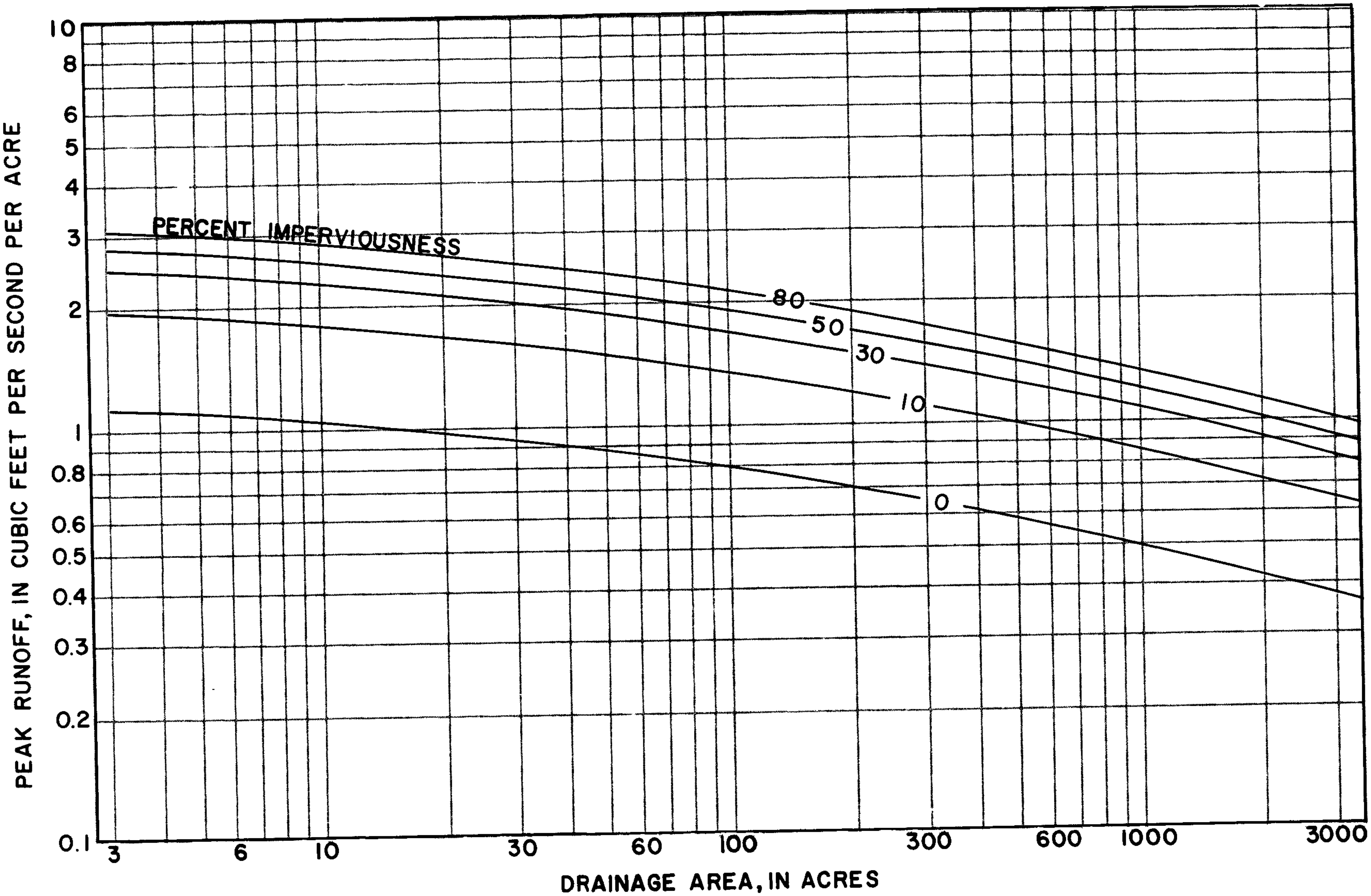


Figure 22.--Runoff per acre for 10-year floods.

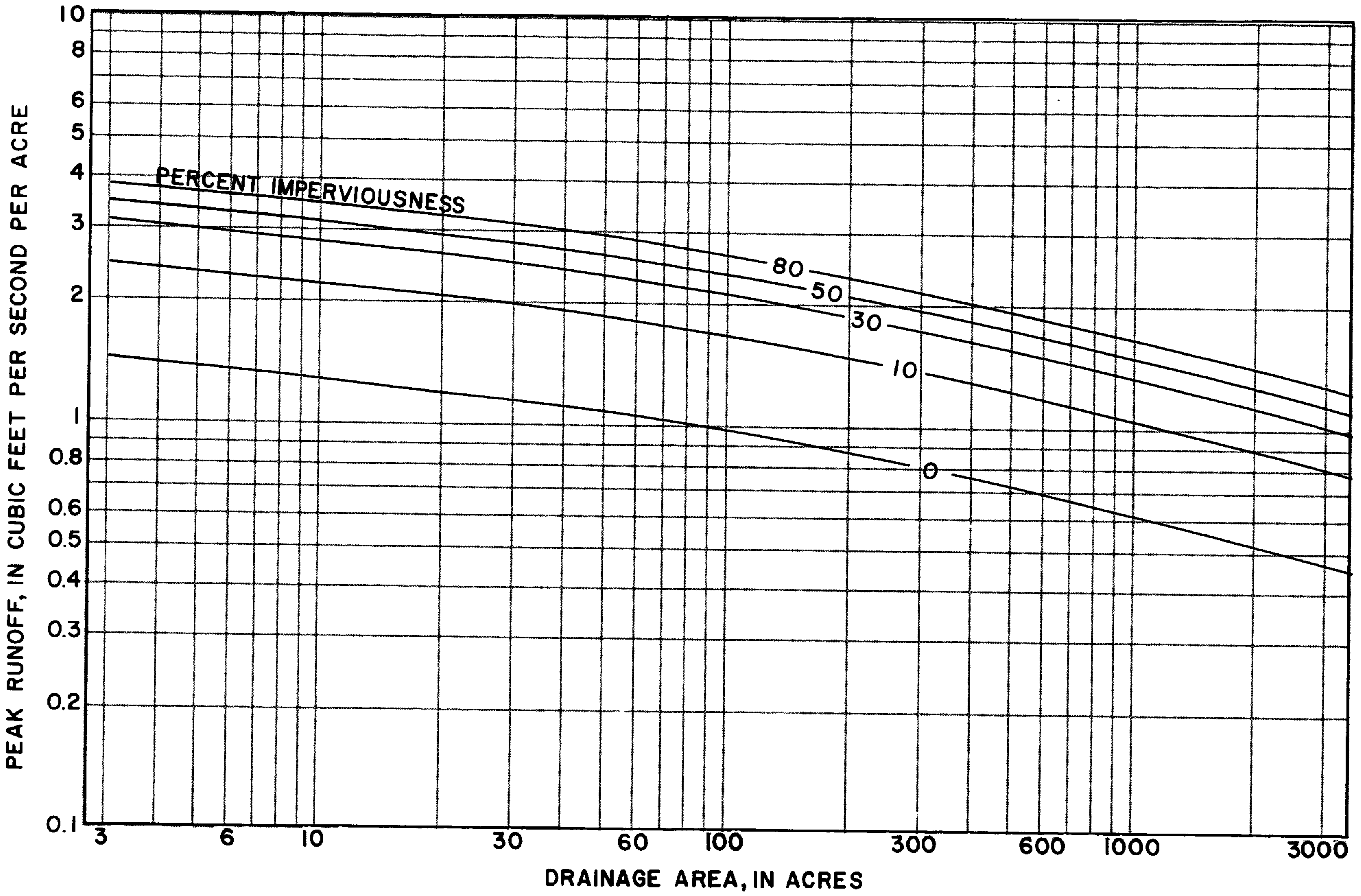


Figure 23.--Runoff per acre for 25-year floods.

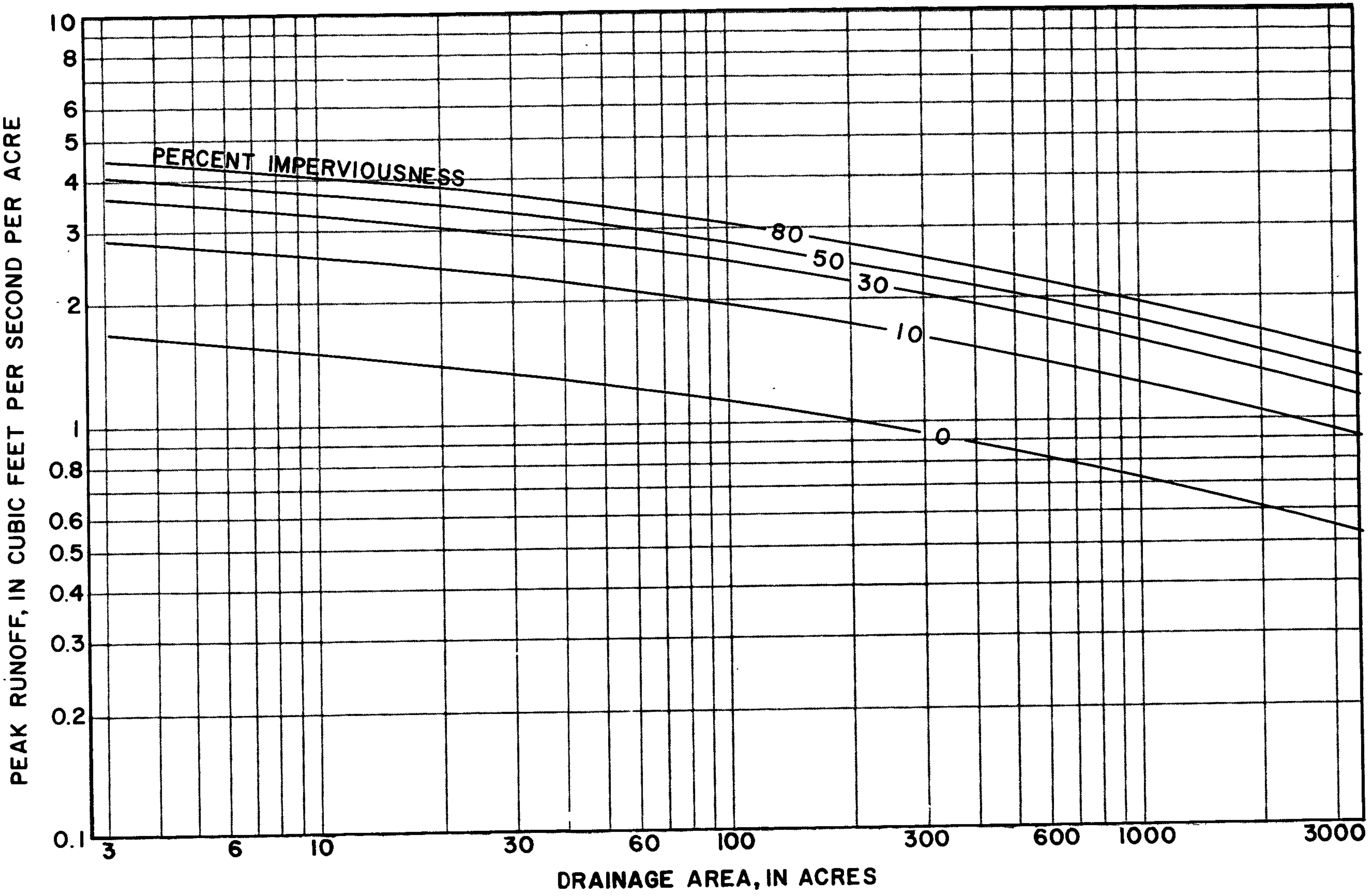


Figure 24.--Runoff per acre for 50-year floods.

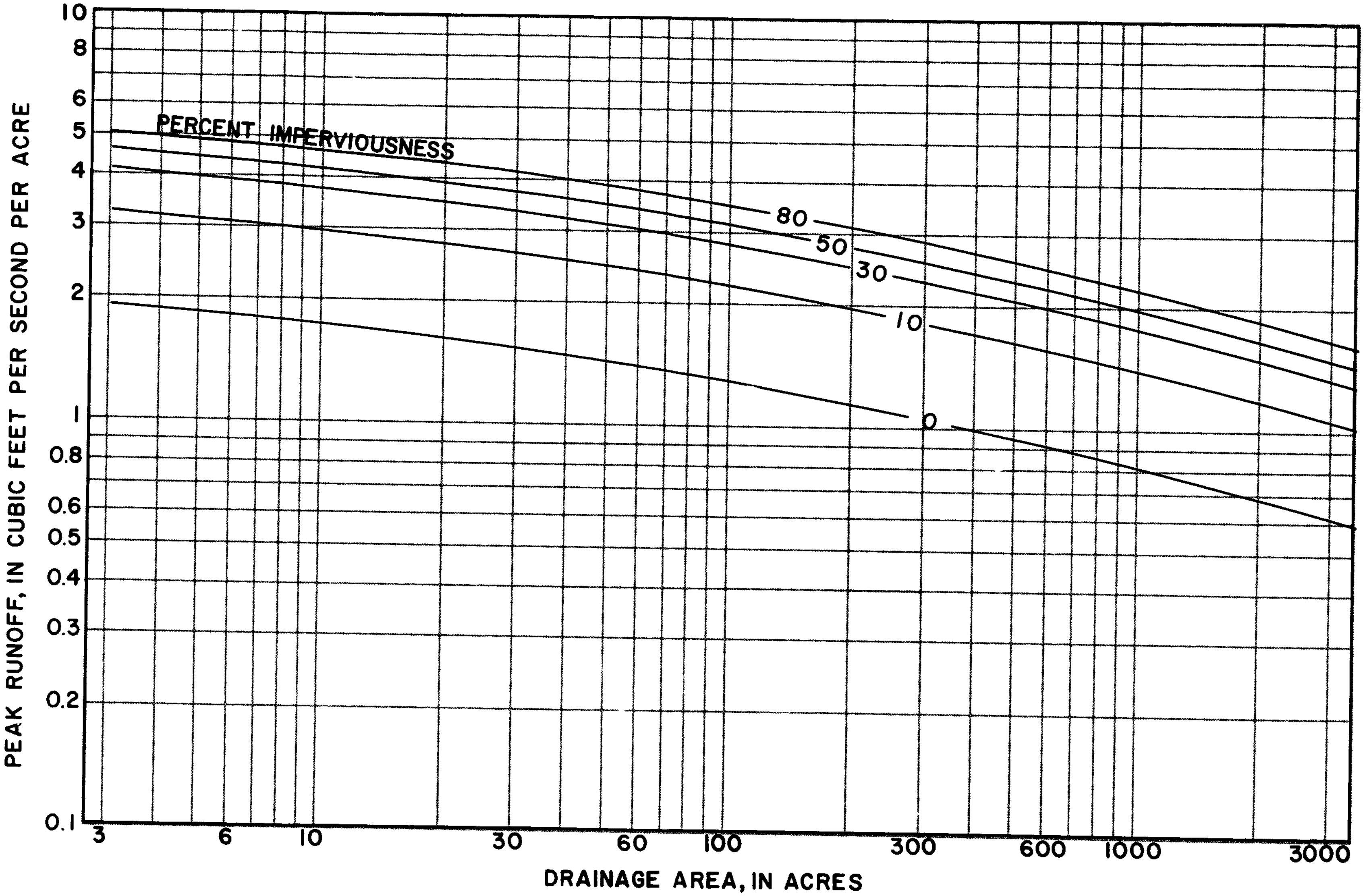


Figure 25.--Runoff per acre for 100-year floods.

SUMMARY AND CONCLUSIONS

The interpretive analysis presented in this report provides a relation defining flood magnitude as a function of drainage area, percent imperviousness of the basin, and the maximum two-hour storm rainfall. This relation was determined by a multiple regression analysis of 59 flood discharges from 19 basins, and the associated basin and rainfall characteristics. The relation was then used with historical rainfall data at Memphis, to synthesize annual flood peaks for various sizes of drainage basins and various degrees of basin imperviousness. From these annual peaks, frequency relations applicable for the Memphis area, were defined and presented for recurrence intervals ranging from 2 to 100 years.

The analytic results are considered as reliable as any that could be obtained by other available methods for predicting flood magnitude on small streams in the Memphis area. The reliability of the equation is indicated by the comparisons, both graphical and statistical, of the simulated and observed floods used to develop the relation, by similar evaluation of the split-sample tests, and by the sensitivity analyses showing variations in discharge to variations in the values of the independent variables.

The analysis points out the apparent deficiencies in present design criteria for storm drainage facilities and substantiates the need for a comprehensive streamflow data base for small streams in the Memphis urban area. Such a base will provide the most reliable means of defining flood frequency relations from which more adequate design criteria can be obtained. Data to satisfy these needs are being collected. However, these data will not be of sufficient length for design purpose for many years, and the continued operation of existing streamflow and rainfall stations is imperative in order to satisfy the data requirements.

The collection of several years of additional data will provide the opportunity to use the approach that was initially outlined for the project, and to consider and evaluate factors not included in this analysis. The regression approach presented here, does not consider the effect of antecedent precipitation on flood magnitude. As presented, the result implies a direct dependency of discharge frequency on rainfall frequency although several investigators have shown that the two are independent. The analysis also does not consider other factors that may significantly affect flood magnitude and storm runoff such as variation in soil type, vegetative cover, stream length and shape, channel shape and improvements, and storm sewer facilities. The effect of these factors will be evaluated in subsequent analyses. The effect of antecedent precipitation on flood magnitude can be evaluated by the use of the proposed parametric model. This model (Dawdy, 1972) utilizes daily precipitation in a soil moisture accounting procedure to vary infiltration rates during storms. Other basin parameters, as well as the drainage area, imperviousness, and slope that were evaluated in this investigation, will be included in subsequent analyses to aid in defining a relation that can be more confidently applied to ungaged streams. The additional data collected in the next several years should also reduce extrapolation errors that may be inherrent in the relation defined in this report. The data utilized for calibration in this analyses are for much smaller storms than for those used in synthesis; additional years of gaging should provide data for greater storm magnitudes and thus reduce the amount of extrapolation.

The relative merits of the statistical analytic approach described in this report and the use of a parametric rainfall-runoff model cannot be defined at this time. The parametric model mathematically simulates various physical and hydrologic characteristics of stream basins and thus describes, more realistically, the response of a basin to storm rainfall. The calibrated model can be used to synthesize annual peak flows from which frequency curves for the gaged streams can be defined. Peak flows of equal frequencies can then be related to selected basin and urbanizing characteristics to allow determination of floods for selected frequencies for ungaged basins. Upon completion of such a parametric modeling analysis, the two approaches to flood frequency analysis may be evaluated, and their relative merits and reliabilities defined.

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