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EFFECTS OF URBANIZATION ON FLOODS IN THE DALLAS, TEXAS METROPOLITAN AREA

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16. Abstracts The effects of urbanization on flood characteristics of streams in the Dallas metropolitan area were studied by use of a digital model of the hydrologic system, which was calibrated by using observed data from 19 storms in six basins to estimate peak discharges and flood volumes and to simulate a 57-year record of annual peak discharges in 14 basins. The flood-frequency characteristics were defined by fitting the simulated 57-year record to a log-Pearson Type III distribution.

Regional peak-discharge equations, which can be used to determine the maximum rates of discharge that could be expected to be equalled or exceeded on the average of once in 1.25, 2, 5, 10, 25, and 100 years, were derived from multiple-regression analyses. The relationships among flood frequency, drainage area, and a coefficient of impervious area are given in a nomograph.

The data indicate that in a fully developed residential area, with 37 percent impervious area, the average annual direct funoff is about double that of an undeveloped area.

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COVER--*Aerial view looking north showing flooding of White Rock Creek at Garland Road, September 21, 1964. Spillway of White Rock Lake in background. Photograph by Tom Dillard, Dallas, Texas*

CONTENTS

ILLUSTRATIONS

TABLES

EFFECTS OF URBANIZATION ON FLOODS IN THE DALLAS, TEXAS, METROPOLITAN AREA

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ABSTRACT

The effects of urbanization on flood characteristics of streams in the Dallas metropolitan area were studied by use of a digital model of the hydrologic system. The model was calibrated by using observed rainfall and runoff data from 19 storms in six basins having various degrees of urbanization. The calibrated models were used with a 57-year rainfall record to simulate 57-year records of annual peak discharges in 14 basins. The flood-frequency characteristics were defined by fitting the simulated 57-year records to log-Pearson Type III distributions.

Regional peak-discharge equations, which can be used to determine the maximum rates of discharge that could be expected to be equaled or exceeded on the average of once in 1.25, 2, 5, 10, 25, and 100 years, were derived from multiple-regression analyses. The relationships among flood frequency, drainage area, and a coefficient of impervious area are given in a nomograph.

The analyses indicate that in a fully-developed residential area, the flood peaks will be 1.2 to 1.4 times those from an undeveloped area; and the annual direct runoff will be about double that from an undeveloped area. Data were not sufficient to determine the increase in runoff from a highly industrialized area where the effective imperviousness approaches 100 percent.

INTRODUCTION

Purpose and Scope of the Urban Study

A program to define the effects of urban development on flood characteristics of streams in the Dallas metropolitan area was begun in 1962 by the U.S. Geological Survey in cooperation with the city of Dallas. The purpose of this study was to determine the effects of urbanization on the magnitude and frequency of floods, to establish a regional flood-frequency relationship of sufficient accuracy to be used for the design of drainage systems, and to determine the relative importance of the physical characteristics that influence the hydrology of an urban area.

The analytical methods used encompass general statistical concepts. A digital model of the hydrologic system, which was calibrated by using data from 19 storms in each of six basins, was used to simulate 57-year records of annual peak discharges for 14 drainage basins.

The simulated 57-year records were used to define flood-frequency relations by fitting to the log-Pearson Type III distribution (U.S. Water Resources Council, 1967, p. 7-9). Regional peak-discharge equations, which can be used to determine the maximum rates of discharge that could be expected to be equaled or exceeded at average intervals of 1.25, 2, 5, 10, 25, 50, and 100 years, were derived from multiple-regression analyses.

The general relationships of flood-peak characteristics, drainage area, and degree of urbanization can be used to estimate the effects of varying degrees of urbanization and to estimate flood-peak characteristics at ungaged sites.

Acknowledgments

The author acknowledges the assistance of Mr. Harry H. Stirman, Director of the Department of Public Works of the city of Dallas, and Mr. Donald H. Deere, Drainage Engineer, Dallas Department of Public Works. Mr. Robert W. Lichty, Research Hydrologist, U.S. Geological Survey, assisted with the development of computer applications and modeling techniques.

Conversion to Metric Units

For those readers interested in using the metric system, metric equivalents of English units of measurements are given in parentheses. The English units used in this report may be converted to metric units by the following conversion factors:

From	Multiply by	To obtain
Square miles (mi^2)	2.590	Square kilometers
Miles (mi)	1.609	Kilometers
Feet (ft)	.3048	Meters
Square feet (ft^2)	.0929	Square meters
Inches (in)	2.54	Centimeters
Cubic feet per second (ft^3/s)	.02832	Cubic meters per second

Physical Setting

The Dallas metropolitan area is in north Texas about 250 miles (400 kilometers) from the Gulf of Mexico (fig. 1). The city has grown rapidly in recent years, and presently (1973) very little land remains undeveloped.

The altitude ranges from about 500 to about 700 feet (150 to 210 meters) above mean sea level. In the upland areas, outcrops of limestone, chalk, and marl are surrounded by a thin mantle of soil. The soil mantle becomes much thicker and more extensive in the lowland areas. The soils are mostly clays that are dark colored and extremely sticky when wet. As with most clays, they show marked changes in volume with changes in the moisture content. During dry periods, numerous cracks develop in the clay and it becomes highly permeable; but almost immediately after wetting, the clay expands and the cracks rapidly close, thereby reducing the permeability. In residential areas and public parks, extensive watering throughout most of the year generally reduces the cracking.

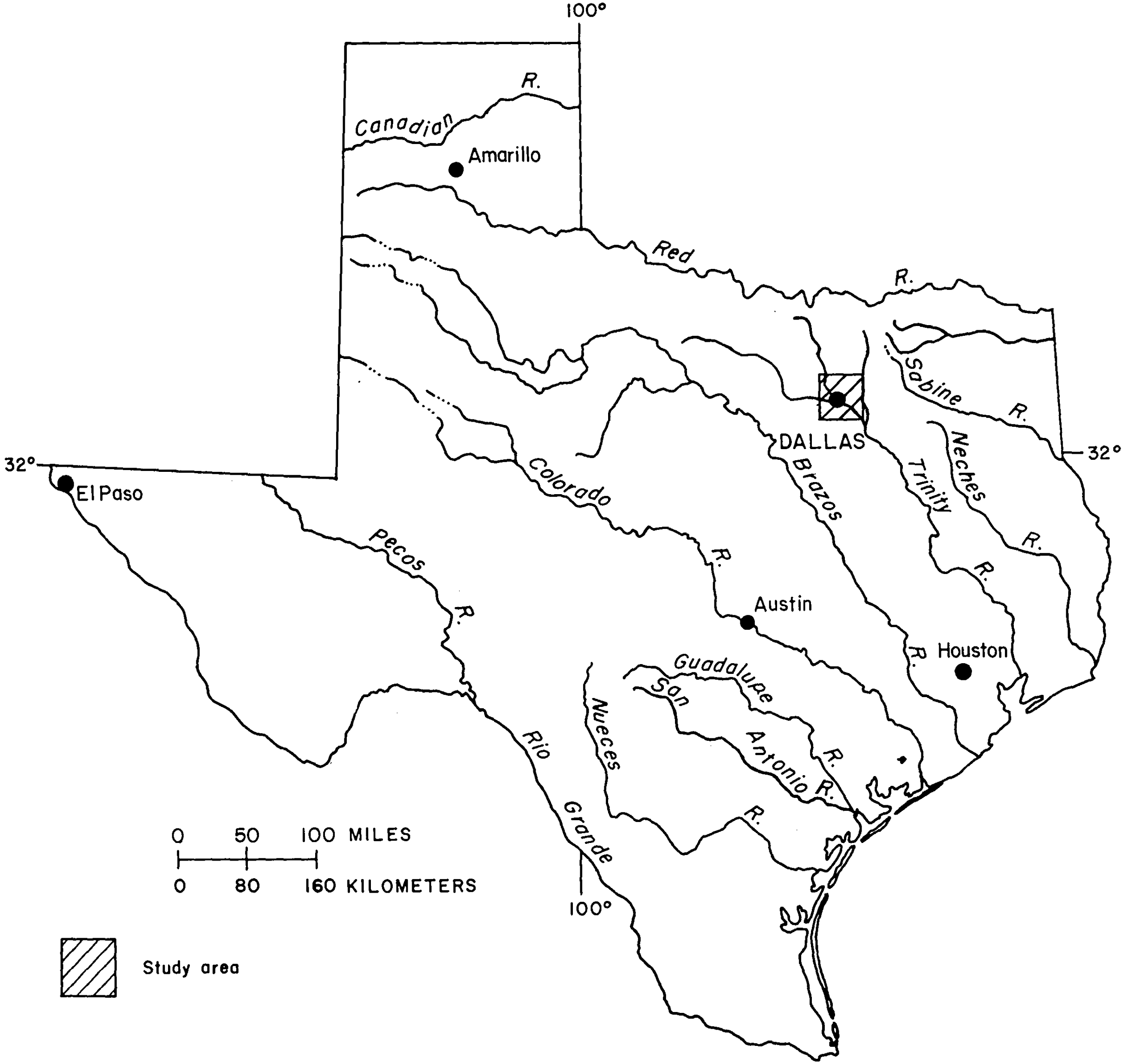


FIGURE 1.-Map of Texas showing the location of the Dallas metropolitan area

The climate of Dallas is generally temperate with hot summers and mild winters. Mean temperatures range from 45°F (7.2°C) in January to 85°F (30°C) in July. The most common storms are thunderstorms that occur frequently in the spring and summer. Long-duration low-intensity storms triggered by southward-moving continental polar fronts occur during the fall and winter. In late summer and early fall, hurricanes moving inland from the Gulf of Mexico cause some of the heaviest rainfall. Individual storms, although most frequent in the spring, may cause serious flooding during any season. Mean annual rainfall at Dallas for 1913-70 was 34.90 inches (88.6 centimeters). Lake evaporation usually exceeds rainfall from late March to early November.

The major stream draining the area is the Trinity River, which divides the city into two parts. The principal tributaries are Joes Creek, Bachman Branch, Turtle Creek, White Rock Creek, Coombs Creek, Cedar Creek, and Fivemile Creek. Most streams and their tributaries have well incised channels with steep banks of limestone, particularly in the upper reaches. The average channel slopes commonly exceed 30 feet per mile (5.7 meters per kilometer).

DESCRIPTION OF THE HYDROLOGIC MODEL

A digital model developed by Dawdy, Lichty, and Bergmann (1972) and modified by Lichty (written commun., 1971) was used to simulate long records of peak discharges under existing conditions of urbanization. The structure of the model is shown by the diagram on figure 2. The input parameters are identified in table 1. Figure 2 shows the general sequence of computations and shows that the output from one component is the input to the next.

The antecedent-moisture accounting component (fig. 2), which is a more sophisticated version of the antecedent-precipitation index (API), measures the effects of antecedent conditions on the infiltration component. The infiltration component is based on an equation described by Philip (1954), in which infiltration rates are computed as a function of soil moisture and rainfall intensity. Infiltration does not occur in impervious areas, but some retention does occur. The model assumes 0.05 inch (0.12 centimeter) of water depth as the maximum retention in an impervious area.

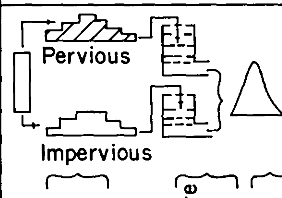
The surface-routing component (fig. 2) is based on unit hydrograph concepts (Sherman, 1932), and assumes a conceptual model composed of linear reservoirs and channels. Rainfall excess is converted into flood hydrographs by procedures representing the effects of varying travel times and reservoir delays, which are derived from distance-area curves. The derivation assumes that travel time and reservoir delay are proportional, but some flexibility is permissible.

ANTECEDENT-MOISTURE ACCOUNTING COMPONENT	INFILTRATION COMPONENT	ROUTING COMPONENT

INPUT DATA

Daily rainfall Daily pan evaporation Initial estimates	Unit rainfall BMS SMS (from moisture accounting component)	Rainfall excesses (from pervious area infiltration and impervious area retention component)

COMPUTATIONAL OPERATION AND PARAMETERS

Saturated-unsaturated soil moisture levels for pervious area	Average pervious area infiltration and impervious area retention	Instantaneous hydrographs from pervious and impervious areas
Parameter EVC RR BMSM DRN	Variable BMS SMS PARAMETER SWF KSAT RGF	 Pervious Impervious Rainfall excesses Lag by distance (time)-area histograms Attenuation by proportionate linear-storage routing Flood hydrograph Parameter TC KSW

OUTPUT DATA

Variable BMS - Base-moisture storage available SMS - Surface-moisture content from infiltration	Rainfall excess	Discharge

FIGURE 2.-Schematic diagram of the hydrologic model

Table 1.--Identification and definition of the parameters used in the digital model

Component	Parameter identifier	Units	Definition
Antecedent-moisture accounting	EVC	--	Pan coefficient that converts pan evaporation to potential evapotranspiration.
	RR	--	Coefficient that proportions daily rainfall into infiltration and surface runoff.
	BMSM	Inches	Maximum effective soil-moisture storage volume at field capacity.
	DRN	Inches per hour	Constant coefficient that controls drainage rate of infiltrated soil moisture.
Infiltration	SWF	Inches	Capillary potential, or soil suction, at wetted front for field-capacity conditions.
	RGF	--	Ratio that varies SWF over the soil-moisture range from wilting point to field capacity.
	KSAT	Inches per hour	Minimum saturated value of hydraulic conductivity used to determine infiltration rates.
Routing	TC	Minutes	Time characteristic for translation of rainfall excess by distance-area histrograms.
	KSW	Hours	Time characteristic for linear reservoir routing.

The model allows for the input of multiple rain-gage data for soil-moisture accounting. With multiple input, pairs of distance-area curves for pervious and impervious areas are required for each subbasin. The combination of distributed moisture accounting (multiple-input) and distributed routing make it possible to simulate the effects of both rainfall variability and urban development.

The measured impervious area was reduced and the pervious area was correspondingly increased to account for the flow from impervious surfaces into pervious areas as described subsequently. This reduced imperviousness is called effective imperviousness.

Model-parameter values are determined by a "hill climbing" optimization technique (Dawdy, Lichty, Bergmann, 1972), which uses bounded parameters (to constrain the parameters within reasonable limits) and an objective function. The objective function is the sum of the squared deviations of the logarithms of observed and simulated peak flows and (or) volumes. The combined objective functions give weight to both the storm peaks and storm volumes, while the single function weights either storm peaks or storm volumes as specified.

The parameters are automatically adjusted to minimize the differences between observed and simulated data, as specified in the objective function. A set of parameters can be greater than, less than, or nearly an average of the "true" values. Because of interactions and assumptions, some error always exists. If the differences between observed and simulated data are assumed to be random, then the standard error of estimate, according to statistical analysis, is a meaningful measure of the error (Riggs, 1968).

The input data required for model calibration are rainfall, pan evaporation, storm rainfall, discharge, initial-parameter estimates, base-flow estimates, and an appropriate number of distance-area curves.

DATA-COLLECTION METHODS AND USE OF DATA

Data for this study were collected during 1962-70 (water years) in seven principal basins and six subbasins (fig. 3), which ranged from rural to fully urbanized.

Urbanization within a basin may occur in the lower or upper parts, and construction and drainage improvement may occur concurrently or at differing times and rates. The states of urbanization that existed in 1968 are assumed as mean conditions for the period of record of rainfall and runoff.

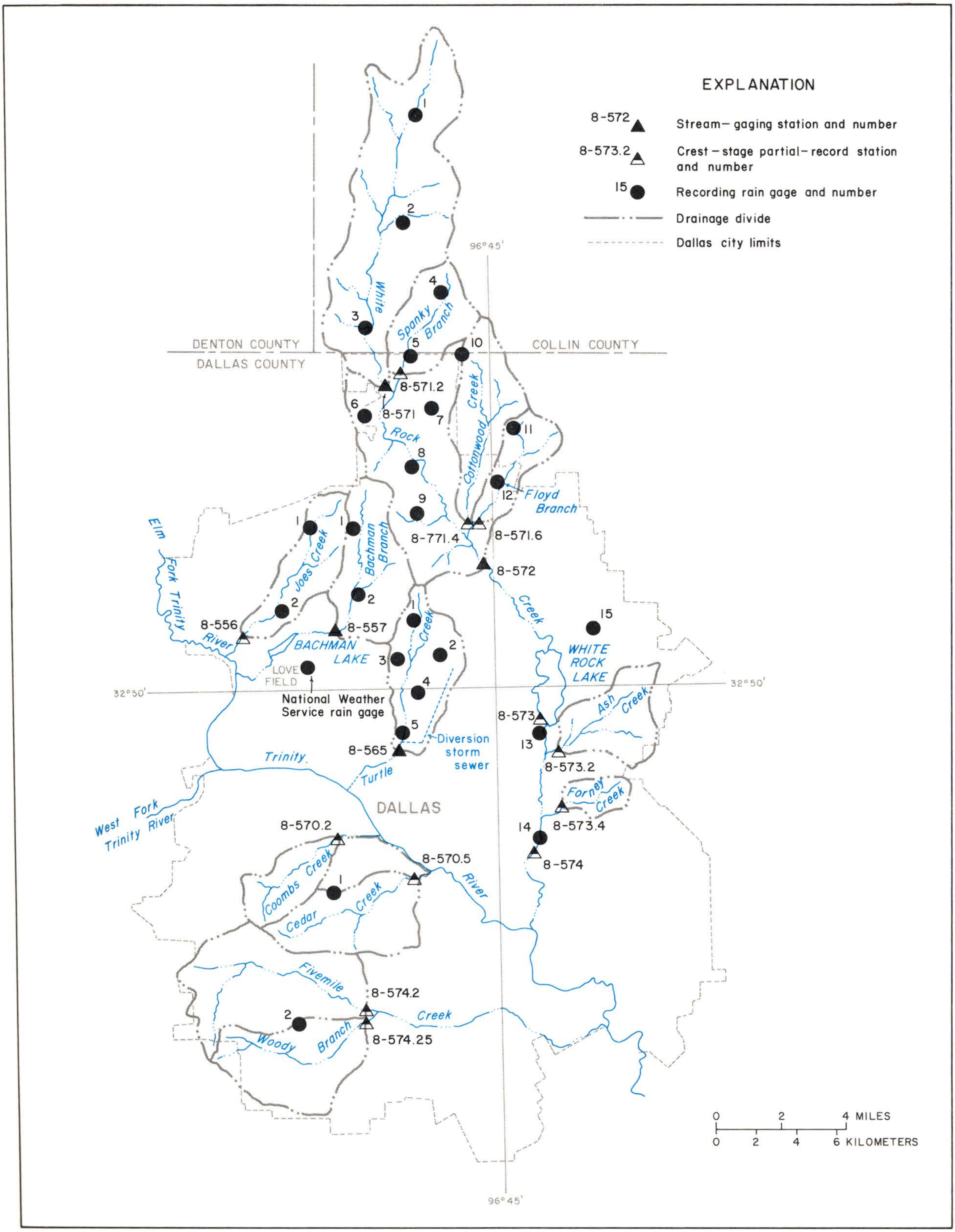


FIGURE 3. -Locations of drainage basins, stream-gaging stations, and rain gages

Drainage Area and Main-Channel Length and Slope

Drainage areas were determined from U.S. Geological Survey topographic maps. Transfer of water from one drainage area to another by storm sewering is insignificant, except in the Turtle Creek Basin where the drainage from Mill Creek was diverted into Turtle Creek before the gaging station was established. Drainage areas are given in table 2.

The main-channel length was calculated as the distance, in miles, along the longest watercourse from the gaging station to the basin divide. The average slope in feet per mile was obtained by dividing the difference in altitude between points located 10 and 85 percent of the distance upstream from the gaging station by the distance in miles between these points (Benson, 1964).

Length-slope ratios are defined as the quotient of the length and square root of the slope; this ratio is usually correlative with lag time. All length and slope data are given in table 2.

Impervious Area

The impervious area of each basin was obtained by: (1) Delineating the different city zoning categories (table 3) and the percentage of development on U.S. Geological Survey maps; (2) assuming that each zoning category had a constant average of impervious area; (3) planimetering the areas of each category and multiplying the areas by the constant average of impervious area for each particular category; and (4) summing the impervious areas of all categories to obtain the total impervious area. The percentage of residential, nonresidential, and total impervious area in each basin (based on mean 1968 conditions) is given in table 2. Table 2 also gives the coefficients of imperviousness and topographic characteristics for each basin.

The coefficient of imperviousness, K, was developed by Carter (1961) to adjust peak discharges due to urban development. The value of K was determined by the formula:

$$K = 1.00 + .015\ I$$

where

I = the percentage of impervious area.

Table 2.--Drainage area, percentage of impervious area, and topographic characteristics of drainage basins in the Dallas metropolitan area

Gaging station (fig. 2)	Basin	Drainage area (sq mi)	Impervious area (percentage)				Coefficient of imperviousness K 3/	Topographic characteristics		
			Residential 1/	Nonresidential	Total	Effective 2/		Length (miles)	Slope (ft/mi)	Length / √slope
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
8-556	Joes Creek	7.51	25	10	35	26	1.52	6.42	31.0	1.15
8-557	Bachman Branch	10.0	24	6	30	22	1.45	6.32	31.6	1.12
8-565	Turtle Creek	7.98	30	17	47	37	1.70	5.90	4/28.1	1.11
8-570.2	Coombs Creek	4.75	33	10	43	31	1.63	4.58	45.2	.68
8-570.5	Cedar Creek	9.42	34	11	45	33	1.68	6.09	38.9	.98
8-571	White Rock Creek above Keller Springs Road	29.4	0	5/1	1	5/1	1.00	13.5	15.2	3.46
8-571.2	Spanky Branch	6.77	0	5/1	1	5/1	1.00	4.88	36.6	.81
8-571.4	Cottonwood Creek	8.50	19	11	30	23	1.45	7.04	32.1	1.24
8-571.6	Floyd Branch	4.17	13	13	26	21	1.39	4.84	38.6	.78
8-572	White Rock Creek above Greenville Avenue	66.4	6	4	10	8	1.15	21.9	12.0	6.33
--	White Rock Creek between Keller Springs Road and Greenville Avenue 6/	37.0	11	3	14	10	-	-	-	-
8-573.2	Ash Creek	6.92	31	7	38	27	1.57	4.44	38.0	.72
8-573.4	Forney Creek	1.84	12	3	15	11	1.22	2.72	56.4	.36
8-574.2	Fivemile Creek	13.2	11	10	21	17	1.32	8.22	32.1	1.45
8-574.25	Woody Branch	11.5	9	4	13	10	1.20	6.12	40.1	.97

1/ Includes all residential zoning categories given in table 3. All other categories are nonresidential.

2/ Calculated as col. 4 x 0.65 + col. 5.

3/ Equation for K = 1.00 + 0.015 I, where I is total basin imperviousness from col. 6.

4/ Based on April 28, 1966, flood profile (Mills and Schroeder, 1969); many channel dams across stream.

5/ Increased to 1.0 percent to account for paved roads and scattered buildings; these are rural basins.

6/ Intervening subbasin; runoff from area used in report.

Table 3.--Zoning categories and average percentage of impervious area

Zoning category designation	Description	Plot size (sq ft)	Average impervious area (percentage)
R-E	Residential estate	>43,560	9
R-1	Residential dwelling	43,560	17
R-1/2	Residential dwelling	21,780	38
R-16	Residential dwelling	16,000	43
R-10	Residential dwelling	10,000	46
R-7.5	Residential dwelling	7,500	46
R-5	Residential dwelling	5,000	50
M-F	Multiple-family dwelling	variable	72
SCH	Schools	variable	35
CH	Churches	variable	85
C	Commercial district	variable	85
SC	Shopping center district	variable	100
IND	Industrial district	variable	72
FW	Freeway	variable	100
OL	Open land 1/	variable	1

1/ Open land in rural areas and public parks increased to 1.0 percent to account for roads, drives, and scattered buildings.

To allow for flow from impervious areas into pervious areas, the impervious area of residential zones was reduced to 65 percent of the total; no reduction was made for nonresidential zones. The basic assumptions were: (1) That an average of 65 percent of the rainfall in residential zones becomes direct runoff in the drainage system, and (2) that 100 percent of the flow from nonresidential zones reaches the drainage system by direct sewering.

Rainfall and Runoff

Peak discharges were measured at four continuous-record gaging stations and 10 crest-stage partial-record stations. Figure 3 shows the locations of the stream-gaging stations. Table 4 gives an identifying map number, basin name, type of station, and availability of streamflow records.

Rainfall amounts and intensities were recorded at rain gages throughout the area. The locations and designation of the rain gages are shown on figure 3; table 4 shows the availability of rainfall records. Table 5 shows the grouping of rain gages and the rainfall-weighting methods used.

Evaporation

Daily evaporation was used to compute daily potential evapotranspiration, which in conjunction with soil moisture, controls the rate of infiltration and ultimately the amount of rainfall excess that appears as surface runoff.

Monthly sunken-pan evaporation records since January 1917 were available from the Texas Agricultural Experiment Station near Denton, which is 30 miles (48.3 kilometers) northwest of Love Field in Dallas. Weather Bureau Class A pan records were available since mid-1953 for another evaporation station located at Grapevine Dam, 18 miles (29.0 kilometers) south of Denton and 14 miles (22.5 kilometers) northwest of Love Field.

For the period of study prior to 1953 (except 1914-16), daily pan evaporation was estimated from monthly records for the Denton station. Daily evaporation for this period was estimated by dividing monthly evaporation by the number of days in each month. For the period 1914-16, prior to the availability of any evaporation data, the average monthly evaporation for the period 1917-53 was used to estimate daily evaporation. After 1953, observed daily pan evaporation at the Grapevine Dam station was used.

Table 4.--Stream-gaging stations and rain gages in the Dallas metropolitan area

Stream-gaging station			Rain gage no. (fig. 3)	Period of record (water years)	
Map no. (fig. 3)	Basin	Type of station		Gaging stations	Rain gages
8-556	Joes Creek	Partial record	1, 2	1964-70	1964-70
8-557	Bachman Branch	Continuous record	1, 2	1964-70	1964-70
8-565	Turtle Creek	Continuous record	1 - 5	1952-70	1962-70
8-570.2	Coombs Creek	Partial record	1/ 1	1965-70	1/ 1969-70
8-570.5	Cedar Creek	Partial record	1/ 1	1965-70	1969-70
8-571	White Rock Creek above Keller Springs Road	Continuous record	1 - 3	1962-70	1962-70
8-571.2	Spanky Branch	Parital record 2/	4, 5, 10	1962-70	1962-70
8-571.4	Cottonwood Creek	Partial record 3/	10 - 12	1962-70	1962-70
8-571.6	Floyd Branch	Partial record 2/	11, 12	1962-70	1962-70
8-572	White Rock Creek above Greenville Avenue	Continuous record	1 - 12	1962-70	1962-70
8-573.2	Ash Creek	Partial record 4/	5/ 14, 15	1963-70	1963-70
8-573.4	Forney Creek	Partial record 2/	14, 15	1963-70	1963-70
8-574.2	Fivemile Creek	Partial record	2	1965-70	1969-70
8-574.25	Woody Branch	Partial record	2	1965-70	1969-70

1/ One rain gage common to stations 8-570.2 and 8-570.5; rain gage with intermittent operation at station 8-570.2 since 1965.

2/ Crest-stage gage only prior to 1968.

3/ Crest-stage gage only prior to 1965.

4/ Crest-stage gage only after 1967.

5/ Rain gage 15 record started in 1965.

Table 5.--Stream-gaging stations, rain gages, rain-gage grouping, and rainfall-weighting methods used in the basins modeled

Gaging station no. (fig. 3)	Basin	Number of subbasin rain gages used	Rain gages for storm period and daily-rainfall definition 1/	Rainfall-weighting method
8-556	Joes Creek	2	(1-J, 1-B*), (2-J*)	Thiessen
8-557	Bachman Branch	2	(1-B*), (2-B*, 1-T)	Thiessen
8-565	Turtle Creek	2	(1-T*, 2-T, 3-T), (2-T, 3-T, 4-T*, 5-T)	Thiessen
8-570.2	Coombs Creek	1	2/ (8-570.2*), (5-T)	--
8-570.5	Cedar Creek	1	2/ (8-570.2*), (14-W)	--
8-571	White Rock Creek above Keller Springs Road	1	(1-W, 2-W*, 3-W)	Average
8-571.2	Spanky Branch	1	(4-W, 5-W*, 10-W)	Thiessen
8-571.4	Cottonwood Creek	1	(10-W, 11-W*, 12-W)	Thiessen
8-571.6	Floyd Branch	1	(11-W*, 12-W)	Thiessen
8-572	White Rock Creek above Greenville Avenue	1	(1-W through 12-W)*	Average
-	White Rock Creek between Keller Springs Road and Greenville Avenue	-	(4-W through 12-W)	Average
8-573.2	Ash Creek	1	(14-W, 15-W*)	Thiessen, distance
8-573.4	Forney Creek	1	(14-W*)	--
8-574.2	Fivemile Creek	1	(2-F*)	--
8-574.25	Woody Branch	1	(2-F*)	--

1/ Alphabetic designation with rain-gage number is keyed to location; example, 1-J is gage number 1 in Joes Creek Basin (fig. 2).

2/ Rain gage operated at crest-stage partial-record station.

* Denotes gage used for time distribution of storm-period rainfall within a day and for daily-moisture accounting (single gage, nonweighted) between storm days, except Coombs Creek and Cedar Creek, where gages 5-T and 14-W were used for daily-moisture accounting; White Rock Creek above Greenville Avenue was the average of all available gages.

Because the pan evaporation at Grapevine Dam was used as input to the model during calibration, it was desirable to use the equivalent for the entire period of peak-discharge simulation (1914-70). Ratios of monthly evaporation as measured at Grapevine Dam and at Denton were defined from concurrent periods of observation. The ratios varied from 1.25 in December and January to 1.85 for May through July. These ratios were applied to the records for Denton from January 1917 to October 1953.

Distance-Area Curves

The shape of the runoff hydrograph is dependent upon storage characteristics and travel time. The routing procedure in the hydrologic model transforms rainfall excess from drainage-area increments into discharge by application of proper travel time and reservoir delay.

Travel time was assumed to be proportional to distance. Isochronal lines representing varying travel times were drawn on maps as simple arcs with varying radii, with some weight given to abrupt changes in the shape of the basins and directions of the channel. The effects of storm sewering on travel time were not investigated. Isochronal subareas were measured, accumulated, and plotted against channel distances to derive the distance-area curves.

COMPARISON OF OBSERVED AND COMPUTED PEAK DISCHARGES

Observed peak discharges and flood volumes were compared with those computed by the hydrologic model to determine if the model produced reasonable results. Both the observed and computed data are given in table 6. The results are shown graphically as scatter diagrams for the control basins (those for which observed flood-volume data were available to derive moisture-accounting parameters) and noncontrol basins (those in which moisture-accounting parameters were estimated from the parameters derived for the control basins) in figures 4 and 5, respectively.

For the control basins, the average standard error for simulated flood peaks is 26 percent; for the noncontrol basins, the average standard error for the simulated flood peaks is 46 percent.

EXTENSION OF FLOOD RECORDS IN TIME AT GAGING STATIONS

The long-term rainfall records collected by the National Weather Service since 1914 at Dallas and nearby long-term records of measured or estimated pan evaporation were used as input to the hydrologic model to

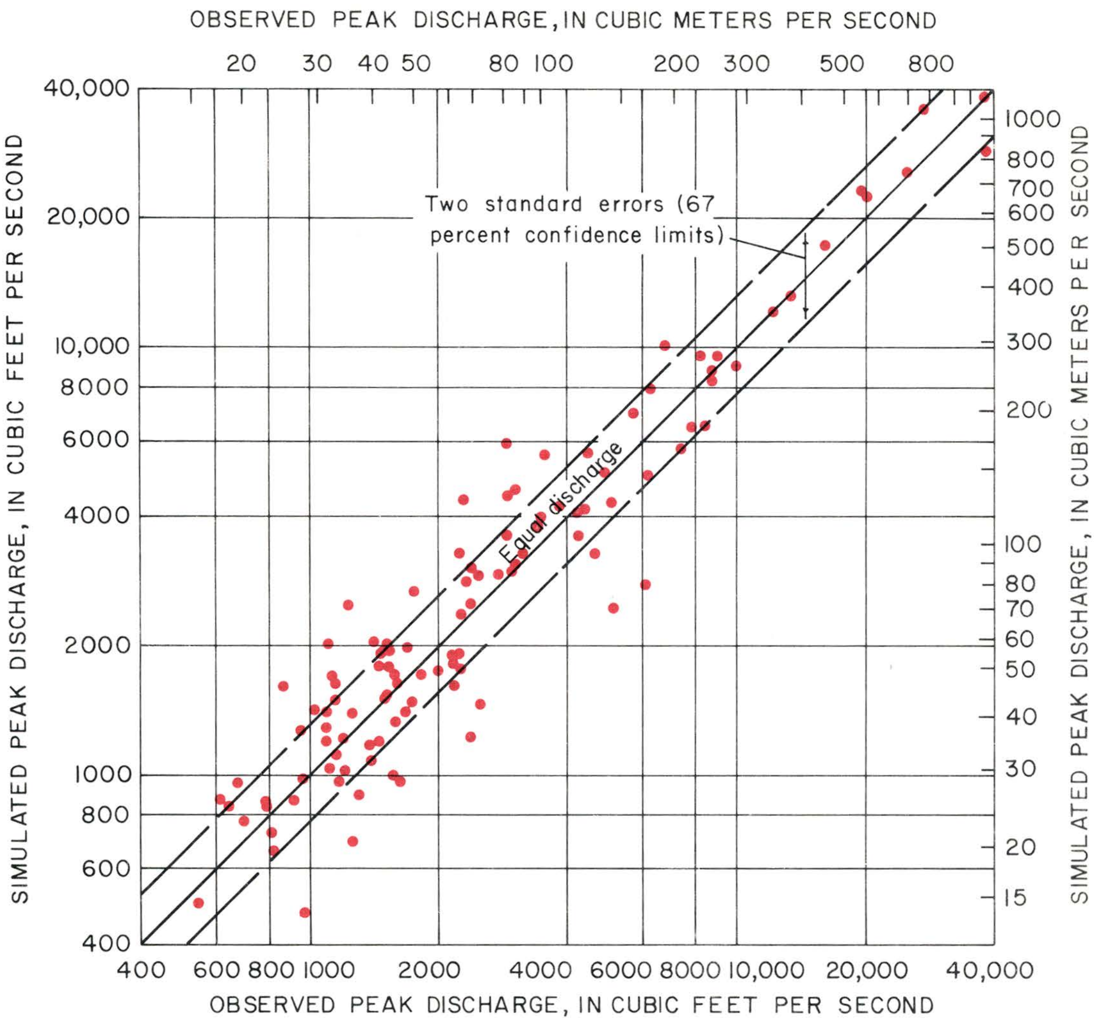


FIGURE 4.-Scatter diagram of observed and computed peak discharges in the

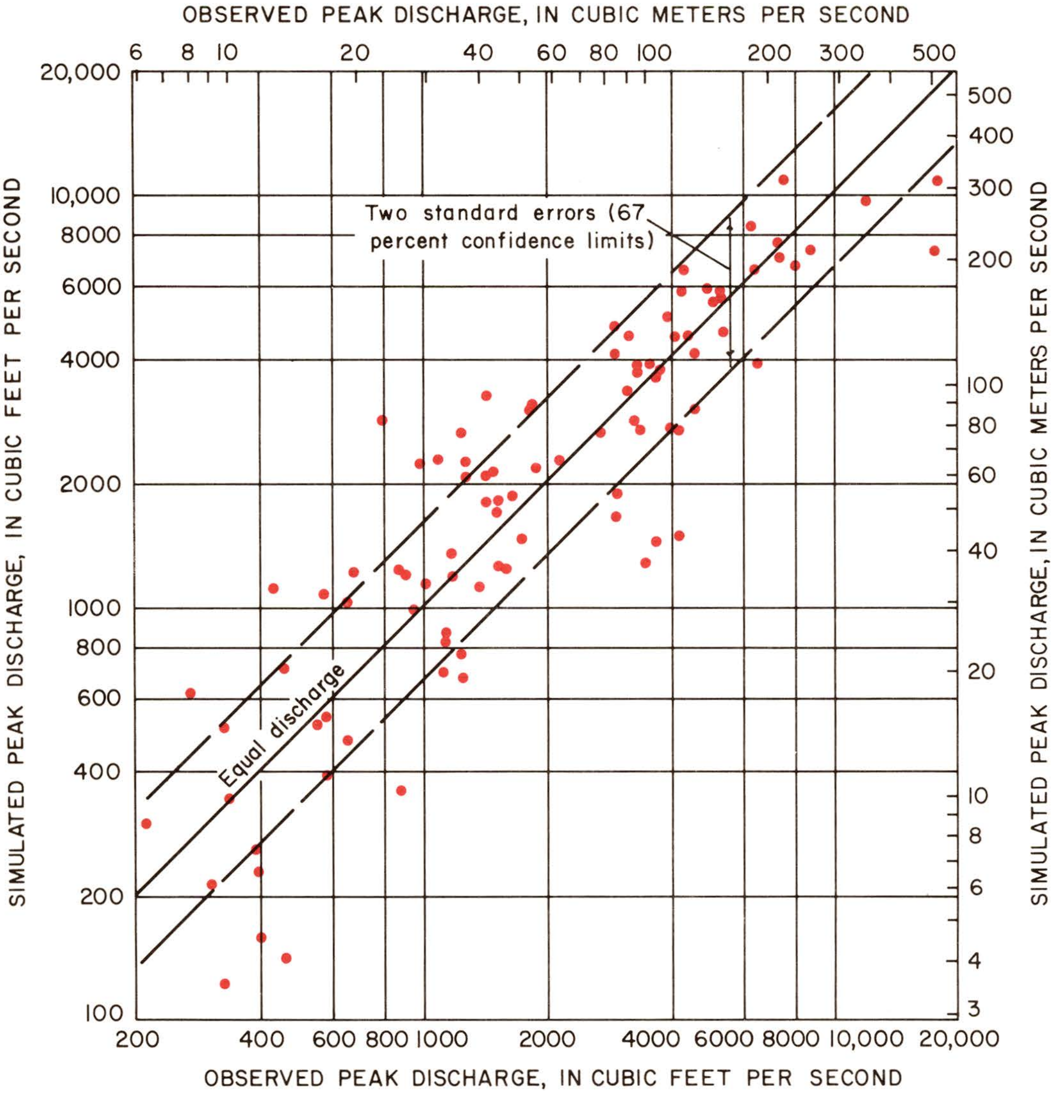


FIGURE 5.-Scatter diagram of observed and computed peak discharges in the noncontrol basins

Table 6.--Storm rainfall and discharge data for Dallas

Storm no.	Date		Storm rainfall (in)	Observed Peak discharge (cfs)	Observed Discharge volume (in)	Simulated Peak discharge (cfs)	Simulated Discharge volume (in)
			8-556. Joes Creek				
1	Apr.	28, 1966	5.03	6,340	4.09	7,670	4.16
2	May	1, 1966	1.96	1,480	1.35	1,830	1.33
3	Apr.	21, 1967	1.40	928	.57	826	.50
4	Oct.	7, 1967	1.55	798	.59	804	.50
5	Mar.	20, 1968	2.17	1,230	1.76	1,160	1.30
6	May	13, 1968	1.23	1,140	.86	1,610	.79
7	Aug.	13, 1968	2.79	1,490	.91	1,850	1.38
8	Oct.	9, 1968	2.18	1,520	.98	1,960	1.00
9	*May	6, 7, 1969	4.90	2,350	2.09	4,240	3.60
10			.80	1,090	.84	1,150	.65
11	*Oct.	12, 1969	2.14	835	.66	634	.72
12			1.41	1,310	.96	863	.65
13	Mar.	3, 1970	1.03	1,200	.70	937	.59
14	*Apr.	25, 1970	.98	793	.37	451	.31
15			1.37	1,460	.72	1,140	.73
16	May	26, 1970	1.76	1,040	.54	1,600	.76
17	May	30, 1970	2.52	1,780	1.12	2,570	1.34
18	Aug.	19, 1970	1.95	1,160	.75	1,430	.75
19	Sept.	2, 1970	2.04	1,640	1.10	1,620	1.28
20	Sept.	23, 1970	1.79	1,410	.88	1,030	.75

Note: For stations 8-556, 8-557, 8-565, 8-570.2, 8-571, and 8-572, observed and simulated discharge data do not include estimated base flow; for other stations, observed discharge data include base flow, and simulated discharge data do not. Asterisk denotes analysis as a complex storm.

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed Peak discharge (cfs)	Observed Discharge volume (in)	Simulated Peak discharge (cfs)	Simulated Discharge volume (in)
			8-557. Bachman Branch				
1	Sept.	22, 1964	1.63	3,030	1.01	2,990	0.98
2	Feb.	8, 9, 1965	3.15	1,650	1.60	933	1.02
3	*May	10, 1965	1.89	2,030	.58	1,690	.56
4			2.27	5,070	1.68	4,150	1.08
5	Feb.	9, 1966	2.05	709	.54	742	.50
6	Apr.	28, 1966	5.21	16,000	4.41	16,900	4.74
7	*June	17, 1966	1.53	1,100	.32	1,340	.45
8			.34	560	.10	472	.12
9	Oct.	4, 1966	1.45	684	.26	901	.35
10	May	31, 1967	1.63	819	.43	697	.43
11	Oct.	7, 1967	1.52	869	.25	1,530	.43
12	Mar.	20, 1968	1.96	1,630	1.51	1,580	1.27
13	May	13, 1968	1.14	1,600	.58	1,270	.36
14	June	24, 1968	1.91	1,170	.53	1,060	.50
15	Oct.	9, 1968	1.81	1,240	.38	2,400	.60
16	*May	6, 7, 1969	5.16	8,350	2.89	8,070	3.05
17			.72	1,120	.34	1,940	.45
18	*Apr.	25, 1970	.96	967	.22	941	.29
19			1.41	2,820	.76	2,840	.74

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed		Simulated	
				Peak discharge (cfs)	Discharge volume (in)	Peak discharge (cfs)	Discharge volume (in)
			8-565. Turtle Creek				
1	Apr.	30, 1962	1.68	3,040	0.94	4,490	1.03
2	July	27, 1962	4.47	4,630	2.29	5,200	2.26
3	Apr.	28, 1963	1.99	4,280	1.89	3,490	1.29
4	June	16, 1963	1.38	1,160	.35	1,560	.52
5	Feb.	9, 1966	2.18	1,280	.87	1,350	.81
6	*Apr.	28, 29, 1966	3.77	12,200	3.77	12,000	3.28
7			1.04	2,350	1.04	2,740	.78
8			.99	2,010	.99	2,640	.75
9	June	17, 1966	1.36	966	.42	1,220	.51
10	Apr.	21, 1967	1.61	1,780	.61	1,410	.61
11	Apr.	22, 1968	1.20	2,250	.53	1,850	.46
12	May	13, 1968	1.46	3,210	.85	3,180	.87
13	June	24, 1968	2.31	1,540	.81	1,460	.88
14	Oct.	9, 1968	1.53	1,540	.47	1,870	.58
15	*Jan.	29, 30, 1969	1.57	2,150	.59	1,830	.58
16			.69	1,230	.34	985	.27
17	May	4, 5, 1969	2.37	2,200	1.03	1,740	.95
18	May	6, 7, 1969	5.59	8,830	4.44	8,530	4.73
19	*Apr.	25, 1970	.88	623	.24	838	.32
20			1.20	2,430	.73	1,190	.50
21	Aug.	19, 1970	1.85	1,420	.42	1,960	.70

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed Peak discharge (cfs)	Observed Discharge volume (in)	Simulated Peak discharge (cfs)	Simulated Discharge volume (in)
			8-570.2. Coombs Creek				
1	*May	10, 1965	1.60	1,120	0.59	989	0.55
2			2.65	4,250	2.09	3,970	1.68
3	*Sept.	21, 1965	1.90	800	.70	818	.63
4			1.53	1,470	.61	1,740	.75
5	*Apr.	29, 1966	1.52	2,410	.89	2,950	1.08
6			.84	1,120	.47	1,230	.50
7	June	12, 1966	1.98	1,730	.57	1,910	.73
8	Apr.	21, 1967	2.00	1,440	.73	1,120	.68
9	June	12, 1967	1.20	1,570	.58	949	.38
10	May	11, 1968	1.20	1,520	.63	1,720	.68
11	May	13, 1968	1.60	2,490	1.12	2,830	1.08
12	June	16, 1968	2.99	2,900	1.02	3,510	1.53
13	Aug.	13, 1968	1.70	2,190	.86	1,560	.60
14	Jan.	29, 1969	1.88	1,530	.66	1,480	.67
15	May	6, 1969	5.66	2,960	3.65	4,340	3.83
16	*Oct.	12, 1969	2.75	2,460	1.19	2,410	1.10
17			1.50	1,280	.58	664	.58
18	May	30, 1970	2.75	2,340	1.97	2,320	1.69

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed Peak discharge (cfs)	Observed Discharge volume (in)	Simulated Peak discharge (cfs)	Simulated Discharge volume (in)
			8-570.5. Cedar Creek				
			(Storms nos. 4, 5, 9, and 11 discharge affected by Trinity River backwater)				
1	*May	10, 1965	1.60	1,250	-	2,060	0.65
2			2.65	7,320	-	7,050	1.84
3	May	15, 1965	2.11	4,180	-	5,940	1.54
4	*Apr.	28, 29, 1966	2.52	6,260	-	6,640	1.89
5			2.38	3,890	-	5,060	1.74
6	June	12, 1966	1.98	3,680	-	3,600	.86
7	Apr.	21, 1967	2.00	2,140	-	2,290	.77
8	June	12, 1967	1.20	1,400	-	1,780	.43
9	May	13, 1968	1.60	2,920	-	4,850	1.17
10	June	16, 1968	3.65	7,500	-	11,000	2.58
11	May	6, 7, 1969	5.65	4,250	-	6,600	3.33
12	Oct.	12, 1969	4.25	5,290	-	4,670	1.92
13	May	30, 1970	3.00	3,910	-	2,730	1.30

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date	Storm rainfall (in)	Observed: Peak discharge (cfs)	Observed: Discharge volume (in)	Simulated: Peak discharge (cfs)	Simulated: Discharge volume (in)
	8-571. White Rock Creek above Keller Springs Road (Records for storm no. 1 partly estimated)					
1	Sept. 20, 21, 1964	14.12	37,900	11.23	28,400	12.09
2	Sept. 27, 1964	1.81	3,450	.81	3,690	.74
3	May 10, 1965	3.54	5,710	1.09	6,820	1.26
4	May 27, 1965	2.39	4,560	.95	5,460	1.06
5	*Apr. 28, 29, 1966	4.15	9,000	2.67	9,200	3.18
6		2.77	7,900	2.25	6,280	1.72
7	Apr. 25, 1967	1.13	1,710	.24	1,350	.23
8	Mar. 20, 1968	3.56	6,200	2.26	4,870	1.66
9	June 24, 1968	1.74	653	.16	802	.15
10	Jan. 29, 30, 1969	2.34	1,860	.49	1,660	.44
11	May 6, 7, 1969	3.45	8,280	1.82	9,270	2.04
12	May 17, 1969	1.89	3,590	.84	5,450	.95
13	May 26, 1969	1.27	3,000	.40	2,880	.47
14	Apr. 25, 1970	2.77	4,930	.91	4,910	.90
15	June 1, 1970	1.37	3,590	.57	3,880	.69

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed Peak discharge (cfs)	Observed Discharge volume (in)	Simulated Peak discharge (cfs)	Simulated Discharge volume (in)
			8-571.2. Spanky Branch				
1	Sept.	27, 1962	7.35	4,020	-	4,570	3.87
2	Sept.	7, 8, 1962	2.58	1,410	-	2,080	1.36
3	Oct.	8, 1962	3.60	2,880	-	4,190	2.02
4	Oct.	28, 1962	3.48	1,340	-	1,120	1.02
5	Sept.	20, 21, 1964	11.78	7,870	-	6,790	9.93
6	Feb.	8, 9, 1964	3.45	1,120	-	836	.81
7	May	10, 1965	4.65	2,670	-	2,670	1.95
8	Apr.	28, 29, 1966	8.23	5,000	-	5,550	5.91
9	May	30, 31, 1967	3.00	635	-	479	.49
10	Mar.	19, 20, 1968	3.72	1,500	-	1,270	1.62
11	May	11, 1968	.87	424	-	718	.31
12	May	6, 7, 1969	4.20	3,680	-	3,710	2.74
13	May	26, 1969	.91	869	-	357	.16
14	Apr.	25, 1970	2.60	1,630	-	1,880	.95

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed		Simulated	
				Peak discharge (cfs)	Discharge volume (in)	Peak discharge (cfs)	Discharge volume (in)
			8-571.4 Cottonwood Creek				
1	June	25, 1962	2.51	794	-	2,840	0.97
2	July	27, 1962	5.95	5,090	-	5,600	3.68
3	Oct.	8, 1962	5.80	17,400	-	7,270	3.15
4	July	14, 1963	2.28	657	-	1,220	.64
5	Sept.	20, 21, 1964	8.59	6,200	-	8,460	6.13
6	Feb.	8, 9, 1965	3.70	1,180	-	789	1.06
7	May	10, 1965	4.65	4,490	-	3,010	1.88
8	Apr.	28, 29, 1966	9.81	17,600	-	10,800	7.73
9	June	12, 1966	1.58	421	-	1,150	.43
10	May	19, 20, 1967	2.03	2,880	-	1,650	.62
11	June	12, 1967	1.30	562	-	1,100	.36
12	Mar.	19, 20, 1968	3.35	890	-	1,200	1.25
13	Aug.	13, 1968	2.82	1,380	-	3,210	1.16
14	May	6, 7, 1969	4.14	4,530	-	4,150	2.64
15	May	17, 1969	1.57	640	-	1,030	.49
16	Apr.	25, 1970	2.59	1,480	-	1,830	.97
17	May	30, 1970	3.27	3,260	-	3,900	1.44

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed		Simulated	
				Peak discharge (cfs)	Discharge volume (in)	Peak discharge (cfs)	Discharge volume (in)
			8-571.6. Floyd Branch				
1	June	25, 1962	3.12	1,770	-	3,030	1.89
2	July	27, 1962	5.37	3,200	-	2,840	3.41
3	Oct.	8, 1962	6.68	4,850	-	5,970	4.67
4	Sept.	20, 21, 1964	7.80	3,500	-	3,890	4.94
5	Feb.	8, 9, 1965	3.76	1,220	-	676	1.52
6	May	10, 1965	4.75	2,880	-	1,890	2.07
7	Apr.	28, 29, 1966	10.61	8,580	-	7,390	9.68
8	May	19, 20, 1967	2.15	1,000	-	1,150	.78
9	Mar.	19, 20, 1968	3.00	1,110	-	699	1.27
10	Aug.	13, 1968	2.94	965	-	2,220	1.44
11	May	6, 7, 1969	4.07	3,350	-	2,730	3.16
12	Apr.	25, 1970	2.68	1,700	-	1,460	1.33
13	May	30, 1970	3.92	3,100	-	3,360	2.20

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date	Storm rainfall (in)	Observed: Peak discharge (cfs)	Observed: Discharge volume (in)	Simulated: Peak discharge (cfs)	Simulated: Discharge volume (in)
	8-572. White Rock Creek above Greenville Avenue					
1	Nov. 22, 1961	1.83	2,920	0.25	5,770	0.54
2	July 27, 1962	6.22	20,000	3.69	22,000	2.86
3	Oct. 8, 1962	4.51	24,500	1.77	25,700	2.66
4	Sept. 20, 21, 1964	11.07	38,100	8.24	38,700	7.18
5	Sept. 27, 1964	1.52	6,820	.75	9,910	1.17
6	*May 10, 1965	1.53	4,420	.20	4,020	.34
7		2.84	13,400	1.70	12,500	1.33
8	Apr. 28, 1966	4.97	26,900	3.19	36,000	4.18
9	*May 30, 31, 1967	1.33	2,850	.16	1,700	.16
10		1.81	6,250	.87	2,660	.35
11	Mar. 20, 1968	2.20	9,960	1.85	8,840	1.45
12	June 24, 1968	1.92	2,260	.28	3,190	.34
13	Aug. 13, 1968	2.40	8,390	.42	6,320	.55
14	Jan. 30, 1969	.84	2,550	.37	1,410	.11
15	May 7, 1969	4.12	19,500	3.66	22,500	3.06
16	May 17, 1969	1.58	7,440	.96	5,630	.63
17	May 26, 1969	1.05	3,890	.33	4,060	.34
18	June 1, 1970	.84	5,200	.45	2,360	.24

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date		Storm rainfall (in)	Observed		Simulated	
				Peak discharge (cfs)	Discharge volume (in)	Peak discharge (cfs)	Discharge volume (in)
8-573.2. Ash Creek							
1	Feb.	8, 9, 1965	3.56	1,500	-	1,710	1.63
2	May	10, 1965	3.60	3,570	-	1,440	1.52
3	Apr.	28, 29, 1966	7.08	5,190	-	5,860	6.54
4	Apr.	21, 1967	1.96	3,370	-	1,280	.66
5	Mar.	20, 1968	1.67	1,150	-	1,340	.93
6	Apr.	28, 1968	1.03	1,540	-	1,250	.53
7	May	13, 1968	1.18	1,430	-	2,110	.85
8	May	6, 7, 1969	5.20	4,330	-	4,520	3.09
9	Oct.	12, 1969	2.68	850	-	1,220	.94
10	Feb.	24, 1970	1.58	576	-	390	.61
11	Apr.	25, 1970	1.78	1,150	-	1,190	.74
12	May	30, 1970	2.68	1,240	-	2,250	1.24
13	Sept.	1, 2, 1970	2.44	927	-	984	.83

Table 6.--Storm rainfall and discharge data for Dallas--Continued

Storm no.	Date	Storm rainfall (in)	Observed		Simulated	
			Peak discharge (cfs)	Discharge volume (in)	Peak discharge (cfs)	Discharge volume (in)
8-573.4. Forney Creek						
1	Feb. 8, 9, 1965	3.00	390	-	230	0.65
2	May 10, 1965	3.16	566	-	539	1.20
3	Apr. 28, 29, 1966	5.05	1,090	-	2,260	3.89
4	Apr. 21, 1967	1.99	200	-	300	.42
5	Mar. 20, 1968	1.21	394	-	159	.38
6	May 11, 1968	1.40	268	-	625	.54
7	May 6, 7, 1969	4.22	1,130	-	845	1.80
8	Apr. 25, 1970	1.60	318	-	511	.52
9	May 30, 1970	3.09	542	-	521	.98
10	Sept. 1, 2, 1970	2.46	450	-	141	.37

Table 6.--Storm rainfall and discharge data for Dallas--Concluded

Storm no.	Date		Storm rainfall (in)	Observed		Simulated	
				Peak discharge (cfs)	Discharge volume (in)	Peak discharge (cfs)	Discharge volume (in)
			8-574.2. Fivemile Creek				
1	Feb.	14, 1969	1.18	385	-	258	0.20
2	Mar.	14, 15, 1969	1.39	330	-	342	.26
3	May	4, 5, 6, 7, 1969	7.54	11,800	-	9,660	4.97
4	Oct.	12, 1969	4.23	3,100	-	4,560	1.63
5	Apr.	18, 1970	1.50	3,320	-	3,770	.74
6	Apr.	25, 1970	1.77	1,860	-	2,180	.58
7	May	30, 1970	3.30	6,380	-	3,890	1.31
			8-574.25. Woody Branch				
1	Feb.	14, 1969	1.18	318	-	123	.11
2	Mar.	14, 15, 1969	1.39	300	-	211	.15
3	May	4, 5, 6, 7, 1969	7.54	7,160	-	7,760	4.72
4	Oct.	12, 1969	4.23	1,810	-	3,050	1.38
5	Apr.	18, 1970	1.50	1,220	-	2,620	.67
6	Apr.	25, 1970	1.77	4,120	-	1,470	.46
7	May	30, 1970	3.30	4,060	-	2,770	1.12

compute annual peak discharges at 14 stations. Data for the storms for which the flood peaks were computed are given in table 7. Table 8 summarizes the peak discharges computed from the model and gives the rank of each annual peak at each site.

Flood-frequency characteristics for streams in Dallas were defined by fitting the log-Pearson Type III probability distribution to the 57-year (1914-70) simulated annual-peak discharges for each of the 14 gaging stations. In the log-Pearson method, peak discharges for selected recurrence intervals are computed by the equation

$$\log Q_T = M + KS$$

where

Q_T = the peak discharge for a selected recurrence interval (T) in years;

M = the mean of the logarithms of the annual peaks;

K = a Pearson Type III frequency factor expressed in number of standard deviations from the mean; and

S = the standard deviation of the logarithms of the annual peaks.

Table 9 gives 7 T-year (recurrence interval) discharges and statistics obtained from fitting the log-Pearson Type III distribution to simulated annual peak-discharge data for each of the 14 gaging stations. Figures 6 and 7 show the flood-frequency curves plotted by using logarithmic-probability scales.

To ensure that the derived log-Pearson Type III frequency curves fit the observed short-term data, each observed annual peak discharge was plotted at its T-year value, computed from

$$T = \frac{n+1}{m}$$

where

n = the number of years of record; and

m = the numerical rank of the peak discharge.

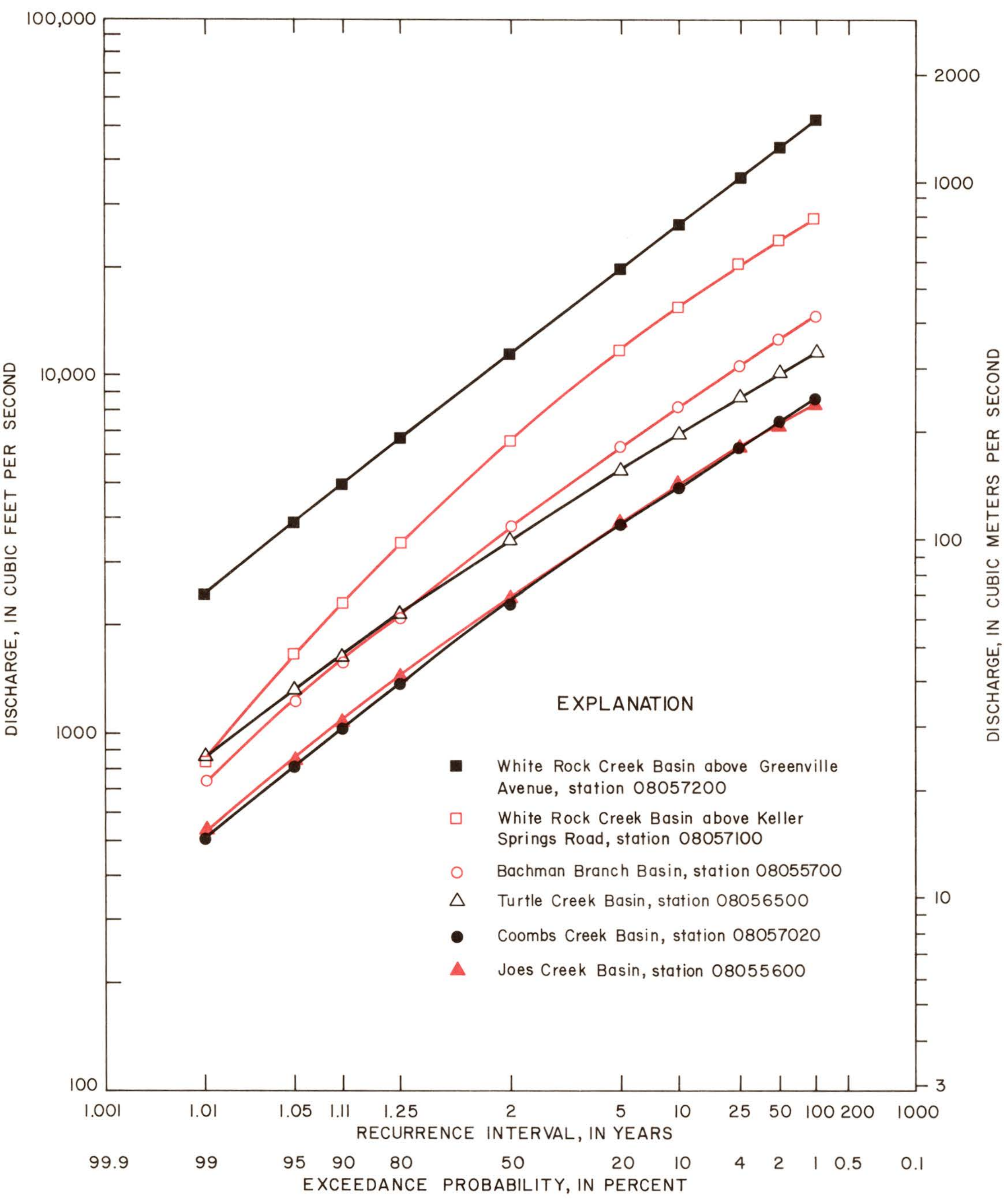


FIGURE 6. - Flood-frequency curves for six gaging stations based on simulated peak discharges for 1914-70

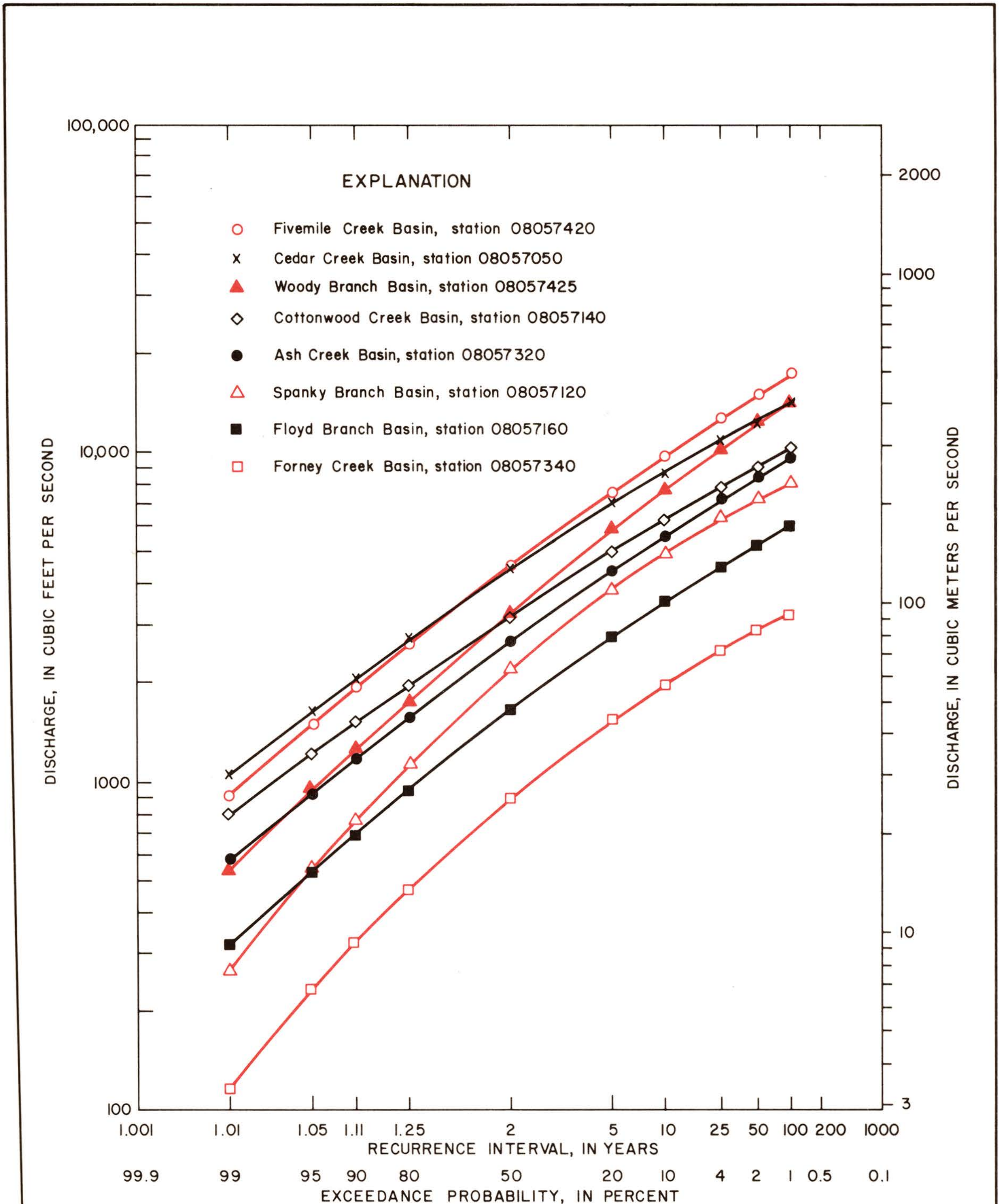


FIGURE 7. - Flood-frequency curves for eight gaging stations based on simulated peak discharges for 1914-70

Table 7.--Storms for which peak discharges were computed from the hydrologic model

Water year	Storm date		Total rainfall (in)
1914	Dec.	2	2.19
	May	4	2.20
	Aug.	25,26	2.06
	Sept.	22	2.04
1915	Aug.	17,18	6.91
		24	2.87
1916	Jan.	26	2.65
	Aug.	5	1.99
1917	Oct.	13	2.79
	May	20	1.38
	Aug.	18	1.65
1918	Apr.	5	3.50
	May	17	2.23
	Aug.	24	2.41
1919	Oct.	26	2.66
	Sept.	21	2.00
1920	Oct.	31	3.47
	Mar.	24	3.97
1921	Apr.	21	1.66
	May	1	1.41
1922	Apr.	3	4.63
		25	4.88
1923	June	2	3.43
		10	3.66
1924	Oct.	14	2.99
	May	26	2.74
1925	May	7	2.89
1925	May	10	1.57
	June	8	2.56
1926	Apr.	10	2.37
	July	7	1.65
	Aug.	17,18	2.79
	Sept.	6	2.28
1927	Mar.	7	3.06
	July	22	1.66
1928	Oct.	1	3.04
	Apr.	5	2.02
1929	May	13	3.45
1930	May	3	1.54
		12	2.49
1931	Sept.	11	2.74
1932	Sept.	3- 5	5.90
1933	Apr.	25	3.40
1934	Sept.	14	4.40
1935	June	14,15	4.70
1936	Sept.	26,27	6.72
1937	June	4	1.29
		16	2.19
	Aug.	23	1.39
	Sept.	6	1.24
1938	Oct.	17	2.70
	Jan.	21	3.00
1939	Apr.	5	2.33
1940	Oct.	9	1.90
1941	June	1, 2	2.76
		27	2.36
1942	Apr.	18-20	3.38
	May	6	2.01
	Sept.	6	2.27
1943	Oct.	15,16	4.50
1944	Mar.	18,19	2.89
	Apr.30,May 1		3.44
	July	12	2.21
1945	July	5	5.34
1946	May	28,29	6.24
1947	Nov.	2	4.83
	Aug.	26,27	9.45
1948	June	28	2.93
1949	Jan.	24	4.88
	May	16,17	5.46
1950	Oct.	24	2.99
	May	1	1.82
1951	June	2	3.22
	Sept.	12	2.38
1952	May	17	2.21
1953	Apr.	23	1.53
		28	2.42
1954	Oct.	25	1.48
	Apr.	11,12	2.31
	May	10-12	4.43
1955	May	19	1.31
1955	June	4	1.51
1956	Apr.	29	2.24
	May	1	2.20
1957	Mar.	31	2.89
	Apr.	26	5.09
	May	23	3.38
1958	Mar.	29	3.05
	Apr.	26	3.39
1959	July	19	1.53
	Sept.	28	1.96
1960	Oct.	1	6.30
	July	13	4.13
1961	Sept.	12	4.02
1962	July	25-27	8.47
1963	Oct.	8	4.92
1964	Sept.	20-22	7.51
1965	May	10	2.63
	Sept.	21	3.45
1966	Apr.	28	3.61
1967	Apr.	21	1.76
1968	May	12,13	1.73
	Aug.	13,14	2.48
1969	Oct.	9	2.44
	May	6, 7	5.43
1970	Oct.	12	4.39
	May	30	1.96
-	---		-

Table 8.--Summary of simulated peak-discharge data

Water year	Joes Creek (No. 8-556)		Bachman Branch (No. 8-557)		Turtle Creek (No. 8-565)		Coombs Creek (No. 8-570.2)		Cedar Creek (No. 8-570.5)		White Rock Creek (No. 8-571)		Spanky Branch (No. 8-571.2)	
	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank
1914	3,080	21	4,770	17	3,500	27	2,140	31	4,590	30	6,260	30	2,250	30
15	2,210	33	1,930	48	2,350	44	754	55	2,530	47	5,010	38	1,350	43
16	2,400	30	3,320	33	4,580	17	3,130	21	5,480	18	7,290	27	2,850	21
17	1,720	41	3,230	36	2,900	34	2,000	35	3,910	35	4,410	39	1,660	38
18	1,400	44	2,600	42	2,490	42	1,690	40	3,280	42	3,440	46	1,290	45
19	3,380	16	4,120	24	4,340	19	2,280	27	5,330	21	8,050	24	2,720	24
1920	4,160	11	7,080	10	5,220	13	3,420	16	6,860	13	11,700	12	3,630	13
21	966	52	1,460	51	1,640	51	1,100	51	2,090	51	1,650	54	613	52
22	5,210	5	7,600	9	5,160	15	3,240	19	7,760	11	13,100	11	3,690	11
23	3,440	13	4,060	25	5,830	12	3,650	12	6,670	14	9,250	17	3,410	15
24	1,740	40	2,930	39	2,760	37	1,600	42	3,510	38	4,300	41	1,430	41
25	2,440	28	4,600	19	4,300	20	2,740	22	5,480	19	6,570	29	2,510	26
26	1,320	47	2,200	46	2,380	43	1,570	44	3,000	45	3,700	42	1,460	40
27	2,460	27	3,300	34	3,200	29	2,030	33	4,650	28	7,560	26	2,450	27
28	1,330	46	2,350	44	2,540	41	1,580	43	3,160	43	3,180	49	1,000	49
29	5,120	7	8,660	7	7,150	7	6,910	2	9,690	6	14,600	7	5,440	6
1930	3,150	18	3,480	30	4,830	16	3,530	13	5,290	22	9,180	18	3,220	16
31	2,210	32	3,870	27	3,590	26	2,260	28	4,780	27	5,990	32	2,000	31
32	3,390	15	5,730	15	4,070	22	3,320	17	6,160	15	11,400	13	3,580	14
33	2,560	26	4,480	20	3,820	24	2,510	25	5,230	23	7,080	28	2,430	29
34	2,020	36	3,130	37	2,310	45	1,520	45	3,290	41	5,940	33	1,800	36
35	2,720	24	4,310	23	3,290	28	1,990	36	4,930	26	8,330	22	2,430	28
36	1,900	38	2,380	43	2,200	46	1,820	38	3,120	44	5,200	36	1,750	37
37	709	55	1,260	54	1,520	52	1,020	53	1,830	53	1,090	56	417	55
38	2,560	25	5,590	16	5,170	14	3,160	20	5,820	16	8,600	21	3,040	19
39	1,140	50	1,360	52	1,490	53	991	54	1,920	52	2,840	50	818	50
1940	707	56	1,060	56	1,260	56	723	56	1,480	56	1,010	57	317	57
41	3,010	23	4,750	18	2,700	38	1,750	39	4,140	33	5,290	34	1,930	34
42	3,200	17	4,450	21	4,430	18	3,490	15	5,810	17	9,140	20	3,160	18
43	1,610	42	2,880	40	2,580	40	1,650	41	3,310	40	3,680	43	1,260	46

Table 8.--Summary of simulated peak-discharge data--Continued

Water year	Joes Creek (No. 8-556)		Bachman Branch (No. 8-557)		Turtle Creek (No. 8-565)		Coombs Creek (No. 8-570.2)		Cedar Creek (No. 8-570.5)		White Rock Creek (No. 8-571)		Spanky Branch (No. 8-571.2)	
	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank
1944	1,880	39	3,300	35	2,770	36	2,280	26	3,580	37	8,240	23	2,620	25
45	5,820	2	10,640	2	8,130	2	7,700	1	11,400	1	19,500	2	6,480	2
46	5,710	3	9,570	4	8,110	3	5,950	3	11,100	2	19,200	3	6,390	3
47	7,200	1	10,900	1	7,420	6	5,070	7	10,800	3	25,500	1	6,930	1
48	1,460	43	1,340	53	1,360	55	1,120	49	1,780	54	3,530	45	1,060	48
49	4,010	12	6,380	12	6,990	8	4,620	8	7,820	9	13,700	9	4,530	9
1950	3,040	22	3,390	31	4,290	21	2,610	23	4,990	25	9,180	19	2,830	22
51	2,410	29	3,320	32	3,170	30	2,200	30	4,200	32	6,190	31	1,970	32
52	1,320	48	2,240	45	2,030	47	1,340	47	2,730	46	3,240	48	1,200	47
53	1,280	49	2,100	47	1,720	50	1,460	46	2,360	48	3,570	44	1,360	42
54	883	54	1,090	55	1,370	54	1,040	52	1,640	55	1,990	51	560	54
55	895	53	1,590	49	1,750	48	1,190	48	2,200	49	1,710	53	653	51
56	2,240	31	4,060	26	3,020	33	3,280	18	4,630	29	8,040	25	2,720	23
57	4,540	9	8,460	8	6,790	10	4,060	11	7,780	10	13,400	10	4,230	10
58	3,120	19	6,710	11	5,940	11	4,380	10	7,120	12	10,100	15	3,790	12
59	1,400	45	2,660	41	2,600	39	1,850	37	3,410	39	3,430	47	1,300	44
1960	5,190	6	9,480	5	8,010	4	5,800	4	10,700	4	17,600	4	5,620	4
61	673	57	832	57	822	57	459	57	1,020	57	1,520	55	397	56
62	4,650	8	8,840	6	6,900	9	5,630	5	9,610	7	15,500	6	5,130	7
63	3,410	14	6,150	14	3,650	25	2,560	24	5,380	20	11,000	14	3,180	17
64	3,080	20	4,420	22	3,820	23	3,500	14	5,050	24	9,480	16	2,980	20
65	2,060	35	3,720	28	3,030	32	2,250	29	4,130	34	5,220	35	1,940	33
66	5,520	4	10,100	3	9,220	1	5,550	6	10,300	5	15,500	5	5,530	5
67	1,020	51	1,560	50	1,750	49	1,100	50	2,110	50	1,840	52	604	53
68	2,140	34	3,120	38	2,870	35	2,030	34	3,870	36	4,340	40	1,640	39
69	4,500	10	6,310	13	7,710	5	4,490	9	8,430	8	14,600	8	4,630	8
1970	1,910	37	3,610	29	3,060	31	2,110	32	4,230	31	5,020	37	1,900	35

Table 8.--Summary of simulated peak-discharge data--Continued

Water year	Cottonwood Creek (No. 8-571.4)		Floyd Branch (No. 8-571.6)		White Rock Creek (No. 8-572)		Ash Creek (No. 8-573.2)		Forney Creek (No. 8-573.4)		Fivemile Creek (No. 8-574.2)		Woody Branch (No. 8-574.25)	
	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank
1914	3,250	29	1,950	22	13,900	18	3,180	22	1,240	17	5,480	20	4,190	19
15	2,160	45	1,110	42	12,600	22	1,870	42	368	49	3,400	40	2,760	35
16	3,710	22	1,820	26	9,060	37	2,860	28	954	26	4,080	35	2,840	33
17	2,690	35	1,300	35	9,040	38	2,180	36	760	32	3,690	37	2,370	40
18	2,300	42	1,060	44	7,080	46	1,820	43	604	43	3,000	46	1,960	47
19	3,800	19	1,990	21	11,100	32	3,280	20	771	31	4,160	31	2,960	32
1920	5,140	12	2,810	12	23,300	11	4,710	11	1,590	13	8,320	11	6,830	11
21	1,470	51	711	49	4,980	50	1,160	50	490	48	2,190	49	1,450	49
22	5,760	9	3,470	10	30,400	4	5,720	9	1,700	12	10,100	8	7,960	8
23	4,530	15	2,090	19	11,200	31	3,330	19	1,000	24	4,840	24	3,440	24
24	2,480	39	1,240	39	8,480	40	2,150	37	595	45	3,380	41	2,240	42
25	3,790	20	1,900	24	12,200	27	3,100	24	1,110	20	5,410	21	3,660	23
26	2,140	46	996	46	7,470	43	1,690	46	807	30	3,170	43	2,340	41
27	3,380	27	1,920	23	12,400	25	2,960	25	862	29	4,510	28	3,300	27
28	2,210	44	1,000	45	7,290	44	1,750	45	533	46	3,190	42	2,050	45
29	6,570	7	3,480	9	22,400	12	5,530	10	2,340	3	10,800	7	8,170	7
1930	3,590	25	1,800	27	10,300	34	2,890	26	731	33	4,300	29	3,080	31
31	3,370	28	1,670	30	11,300	30	2,830	29	715	37	4,690	26	3,190	29
32	4,570	14	2,450	14	19,400	15	3,950	14	1,220	19	7,160	15	5,570	15
33	3,670	24	1,860	25	13,400	20	3,150	23	1,020	23	5,810	18	4,050	20
34	2,540	37	1,270	38	11,700	28	2,100	39	695	39	3,610	39	2,570	38
35	3,730	21	2,170	18	17,300	16	3,530	18	998	25	6,290	17	4,810	17
36	2,280	43	1,130	41	7,500	42	1,940	41	609	42	3,010	45	2,180	43
37	1,280	54	534	54	3,360	55	889	54	252	52	1,460	54	938	54
38	3,980	18	2,330	16	16,400	17	3,720	16	1,440	14	6,470	16	5,070	16
39	1,360	53	709	50	6,450	47	1,190	49	355	50	2,020	50	1,430	50
1940	1,110	56	456	56	3,010	57	823	55	155	56	1,260	56	721	56
41	2,910	33	1,660	31	12,500	24	2,600	31	1,100	21	5,190	23	3,920	22
42	4,170	16	2,310	17	13,800	19	3,630	17	1,290	16	5,710	19	4,620	18
43	2,360	40	1,230	40	8,830	39	2,140	38	653	40	3,700	36	2,480	39

Table 8.--Summary of simulated peak-discharge data--Concluded

Water year	Cottonwood Creek (No. 8-571.4)		Floyd Branch (No. 8-571.6)		White Rock Creek (No. 8-572)		Ash Creek (No. 8-573.2)		Forney Creek (No. 8-573.4)		Fivemile Creek (No. 8-574.2)		Woody Branch (No. 8-574.25)	
	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank	Discharge (cfs)	Rank
1944	2,490	38	1,280	37	12,600	23	2,360	35	727	34	4,130	33	3,180	30
1945	8,060	2	4,500	2	30,200	5	7,260	2	2,310	4	12,800	3	9,830	3
1946	7,820	3	4,280	4	33,600	2	6,860	4	2,480	2	13,400	2	10,300	2
1947	8,840	1	4,610	1	50,200	1	7,850	1	2,180	6	13,900	1	11,200	1
1948	1,360	52	617	53	4,760	51	1,110	53	226	54	1,680	53	1,120	52
1949	5,720	10	2,820	11	22,100	13	4,480	12	1,710	11	7,580	13	5,850	14
1950	3,690	23	2,070	20	12,300	26	3,230	21	718	36	4,270	30	3,210	28
51	2,970	31	1,520	32	9,860	35	2,500	32	636	41	4,100	34	2,800	34
52	2,000	47	949	47	6,320	49	1,580	47	529	47	2,600	48	1,700	48
53	1,620	48	886	48	6,340	48	1,380	48	700	38	2,730	47	2,080	44
54	1,140	55	466	55	3,480	54	820	56	214	55	1,280	55	828	55
55	1,530	49	665	51	4,260	52	1,130	52	352	51	1,820	51	1,200	51
56	3,240	30	1,690	29	11,300	29	2,760	30	934	27	4,660	27	3,440	25
57	5,700	11	3,490	8	28,700	6	5,830	8	1,950	9	9,320	10	7,510	10
58	4,840	13	2,650	13	20,400	14	4,170	13	1,990	8	7,950	12	6,260	12
59	2,350	41	1,100	43	7,220	45	1,800	44	603	44	3,080	44	1,970	46
1960	7,260	4	3,920	5	25,700	7	6,240	5	2,300	5	11,300	5	8,390	6
61	832	57	360	57	3,360	56	659	57	124	57	999	57	637	57
62	6,850	6	3,700	6	25,700	8	6,120	6	2,030	7	10,900	6	8,620	5
63	4,140	17	2,360	15	25,100	9	3,820	15	1,380	15	7,530	14	5,950	13
64	3,420	26	1,790	28	13,100	21	2,860	27	1,230	18	5,260	22	3,970	21
65	2,880	34	1,460	33	10,500	33	2,480	33	1,030	22	4,810	25	3,320	26
66	7,210	5	4,410	3	31,700	3	6,980	3	2,680	1	12,000	4	9,630	4
67	1,510	50	645	52	4,180	53	1,160	51	248	53	1,770	52	1,040	53
68	2,680	36	1,280	36	8,370	41	2,100	40	720	35	3,680	38	2,750	36
69	6,350	8	3,620	7	24,700	10	6,120	7	1,900	10	9,690	9	7,910	9
1970	2,920	32	1,440	34	9,570	36	2,400	34	887	28	4,150	32	2,730	37

Table 9.--Station flood-frequency characteristics computed from simulated peak discharges

Gaging station No. (fig. 3)	Basin name	Peak discharge, Q_T (cfs) at indicated recurrence interval, T (years) (Number in parenthesis is exceedance probability in percent)							Log-Pearson Type III statistics		
		$Q_{1.25}$ (80)	Q_2 (50)	Q_5 (20)	Q_{10} (10)	Q_{25} (4)	Q_{50} (2)	Q_{100} (1)	Mean, logs	Standard deviation, logs	Skewness logs
8-556	Joes Creek	1,420	2,370	3,820	4,840	6,170	7,180	8,200	3.364	0.256	-0.244
8-557	Bachman Branch	2,130	3,710	6,230	8,070	10,500	12,400	14,400	3.558	.278	-.235
8-565	Turtle Creek	2,110	3,410	5,400	6,820	8,680	10,100	11,600	3.526	.243	-.159
8-570.2	Coombs Creek	1,380	2,330	3,830	4,920	6,370	7,490	8,650	3.360	.264	-.185
8-570.5	Cedar Creek	2,740	4,480	7,040	8,790	11,000	12,700	14,300	3.639	.245	-.292
8-571	White Rock Creek above Keller Springs Road	3,340	6,510	11,700	15,400	20,100	23,600	27,100	3.788	.326	-.468
8-571.2	Spanky Branch	1,140	2,220	3,900	5,030	6,420	7,410	8,340	3.314	.323	-.602
8-571.4	Cottonwood Creek	1,960	3,180	5,010	6,290	7,970	9,240	10,500	3.494	.242	-.194
8-571.6	Floyd Branch	945	1,640	2,730	3,510	4,520	5,280	6,050	3.202	.275	-.294
8-572	White Rock Creek above Greenville Avenue	6,600	11,600	20,000	26,600	35,900	43,600	51,800	4.059	.286	-.044
8-573.2	Ash Creek	1,590	2,700	4,430	5,670	7,300	8,550	9,820	3.422	.265	-.232
8-573.4	Forney Creek	472	900	1,560	2,000	2,540	2,920	3,290	2.924	.313	-.582
8-574.2	Fivemile Creek	2,640	4,580	7,700	9,960	13,000	15,300	17,700	3.651	.277	-.230
8-574.25	Woody Branch	1,790	3,320	5,880	7,810	10,400	12,500	14,600	3.508	.308	-.259

The flood-frequency curves generally fit most of the observed data; however, because of the unusually high floods that occurred during 1962-70, the observed data plotted erratically in the upper parts of the curves. The probability of exceedance of these higher flows is unknown even when adjusted historically (Dalrymple, 1960).

REGIONALIZATION OF FLOOD-PEAK CHARACTERISTICS

Multiple-regression techniques are useful in regionalizing streamflow characteristics because discharge for a given recurrence interval can be related to the physical and (or) climatic characteristics of the basins. The regionalization procedure averages the chance variations from sampling while maintaining the variation due to the basin characteristics. Multiple-regression analysis allows definition of predictive equations in the form

$$\log Q = \log C + a \log X_1 + b \log X_2 + c \log X_3;$$

or in the alternate form

$$Q = C\ X_1^a X_2^b X_3^c$$

where

Q = the discharge of selected frequency (dependent variable);

X_1, X_2, X_3 = the basin characteristics (independent variable);

C = the regression constant; and

a, b, c = regression coefficients.

In the regression process, variables that are the least significant are automatically deleted from the equation. The accuracy of the established equation is measured by the standard error of estimate, which represents the degree to which flood-peak variation is explained.

Flood-Frequency Equations

Initially, 13 independent variables were included in the regression computations for each T-year flood. Results of the analyses showed that three of the independent variables, drainage area, coefficient of imperviousness, and the length-slope factor (table 2), would satisfactorily explain more than 98 percent of the variation in the dependent variable. Benson (1964) showed that after inclusion of three or four independent variables, additional variables usually did not appreciably decrease the regression standard error of estimate.

Drainage area (A), which is the most important variable, accounts for at least 90 percent of the flood-discharge variation and is statistically significant at the 5 percent level for all seven T-year floods. The coefficient of imperviousness (K), which is an index to urbanization, and the length-root slope ratio ($L/\sqrt{S}$), which is an index to lag time, account for another 8 percent of the variation. The length-slope factor is also useful in determining the effect of future straightening of the main channel.

The regional flood-frequency equations, the regression standard errors of estimate, and a minimum combined error are summarized in table 10. These equations were derived using simulated peak discharges based on the same long-term rainfall record for all sites. Therefore, the standard errors of estimate associated with the equations are smaller than the total errors. An estimate of the minimum combined error (table 10) is obtained by combining the regression standard error of estimate with the standard error of the rainfall-runoff calibration.

A flood-frequency curve for an ungaged site can be computed if the variables are determined by the methods used in this report and are not outside the observed range. The best results will be obtained for T-year floods between 1.25 and 50 years for drainage areas between 4 and 15 square miles (12.9 and 38.8 square kilometers), $L/\sqrt{S}$ values between 0.7 and 1.5, and K values between 1.1 and 1.6.

Flood-Frequency Nomograph

To aid in application of the flood-frequency equations, a nomograph (fig. 8) was developed by using simplified equations for the four highest T-year discharges (Q_{10}, Q_{25}, Q_{50}, Q_{100}). The simplification involved deleting $L/\sqrt{S}$ from the equations given in table 10, leaving A, K, and the regression constant C. To prepare the nomograph, it was assumed that the variation of the exponents of the variables was insignificant above the 10-year flood discharge so that average exponents could be used for A and

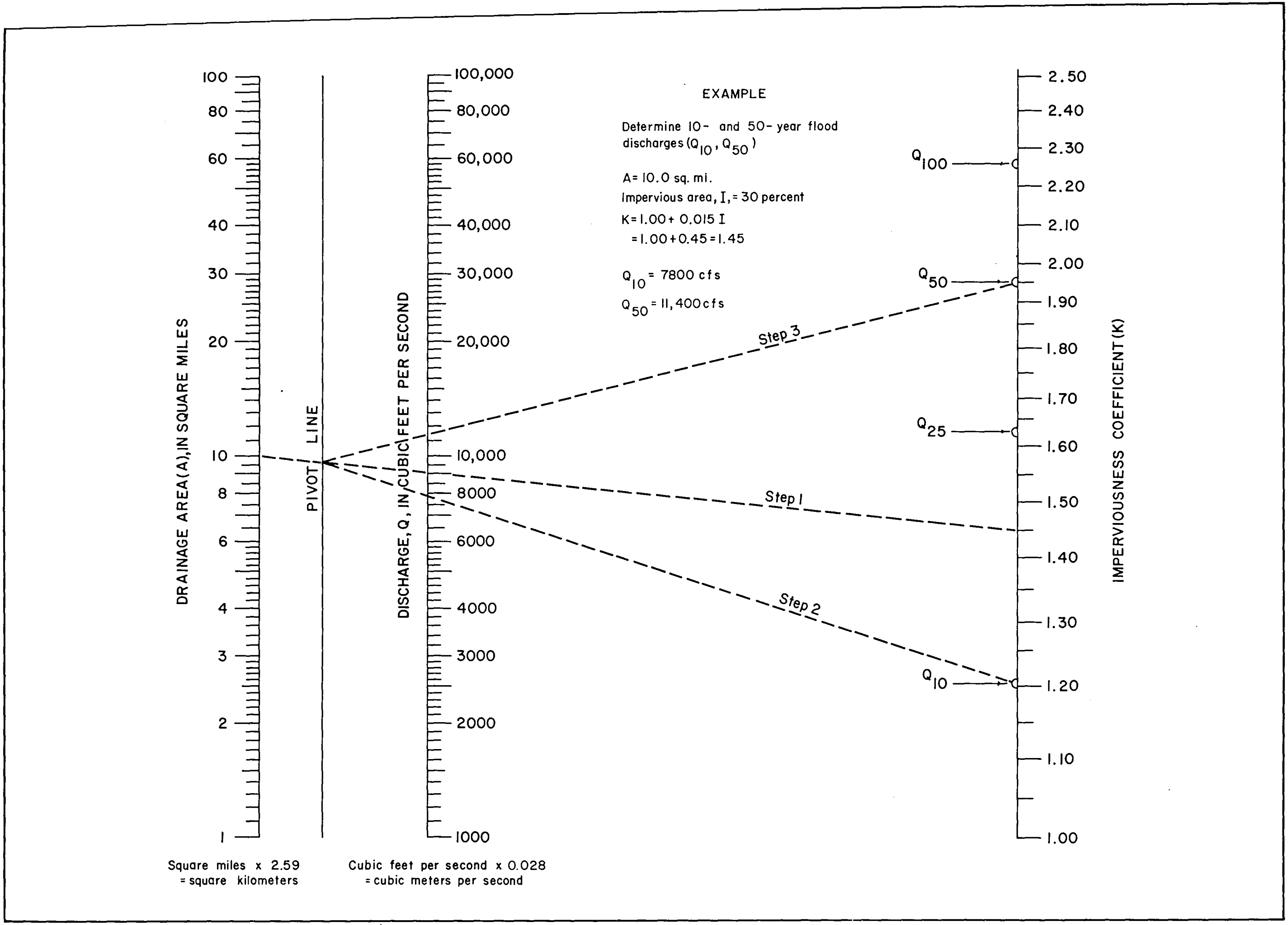


FIGURE 8.-Nomograph showing the relation between flood frequency, drainage area, and a coefficient of imperviousness

Table 10.--Regional flood-frequency equations

Equation for indicated T-year flood discharge (cfs)1/	Standard error of estimate (percent)	Estimate of minimum prediction error (percent)
$Q_{1.25} = 195(A)^{0.88}(L/\sqrt{S})^{-0.13}(K)^{1.02}$	11	30
$Q_{2} = 369(A)^{0.90}(L/\sqrt{S})^{-0.19}(K)^{0.65}$	11	30
$Q_{5} = 621(A)^{0.93}(L/\sqrt{S})^{-0.23}(K)^{0.42}$	10	29
$Q_{10} = 776(A)^{0.95}(L/\sqrt{S})^{-0.25}(K)^{0.35}$	10	29
$Q_{25} = 953(A)^{0.98}(L/\sqrt{S})^{-0.27}(K)^{0.32}$	10	29
$Q_{50} = 1{,}067(A)^{1.00}(L/\sqrt{S})^{-0.28}(K)^{0.32}$	10	29
$Q_{100} = 1{,}172(A)^{1.02}(L/\sqrt{S})^{-0.29}(K)^{0.33}$	10	29

where:

Q = discharge in cubic feet per second (cfs)

A = drainage area in square miles (sq mi)

L = length in miles (mi)

S = slope in feet per mile (ft/mi)

K = coefficient of imperviousness

1/ Cubic feet per second x 0.0283168 = cubic meters per second.

and K. The regression constant C is included in the nomograph by the positioning of each T-year variable. The simplified equations used to determine the nomograph were:

$$Q_{10} = 1150\ A^{.78}\ K^{.36}$$

$$Q_{25} = 1460\ A^{.78}\ K^{.36}$$

$$Q_{50} = 1660\ A^{.78}\ K^{.36}$$

$$Q_{100} = 1870\ A^{.78}\ K^{.36}$$

EFFECT OF URBANIZATION ON FLOOD RUNOFF

Changes in Flood Peaks

Solutions to equations in table 10 can yield the following analogies:

1. When the imperviousness is increased from 0 to 40 percent, Q_2 is increased about 35 percent, Q_{10} is increased about 18 percent, and Q_{50} is increased about 16 percent.

2. When the imperviousness is increased from 0 to 100 percent, Q_2 is increased about 80 percent, Q_{10} is increased about 40 percent, and Q_{50} is increased about 35 percent.

3. When L is decreased and (or) S is increased, the T-year flood is increased.

These analogies are consistent with the results found in hydrologic studies of other urban areas. However, the degree of the effect of impervious area on peak discharge is not as great in Dallas as it is in Houston (Johnson and Sayre, 1973) or other coastal areas. A comparison of the 50-year peak discharges, as a result of urbanization in the Dallas and Houston areas is shown on figure 9. As indicated by figure 9, the channels in Dallas, even before urbanization, are capable of much better conveyance of floods than are the channels in the Houston area even after urbanization. The small increase in the magnitude of peak discharges in the Dallas area after urbanization may be caused by the following factors:

1. The channel conveyance conditions before urbanization are good, with the channels relatively straight and free of vegetation.

2. The soil cover in the Dallas area is thin.

3. Both side and channel slopes are relatively steep.

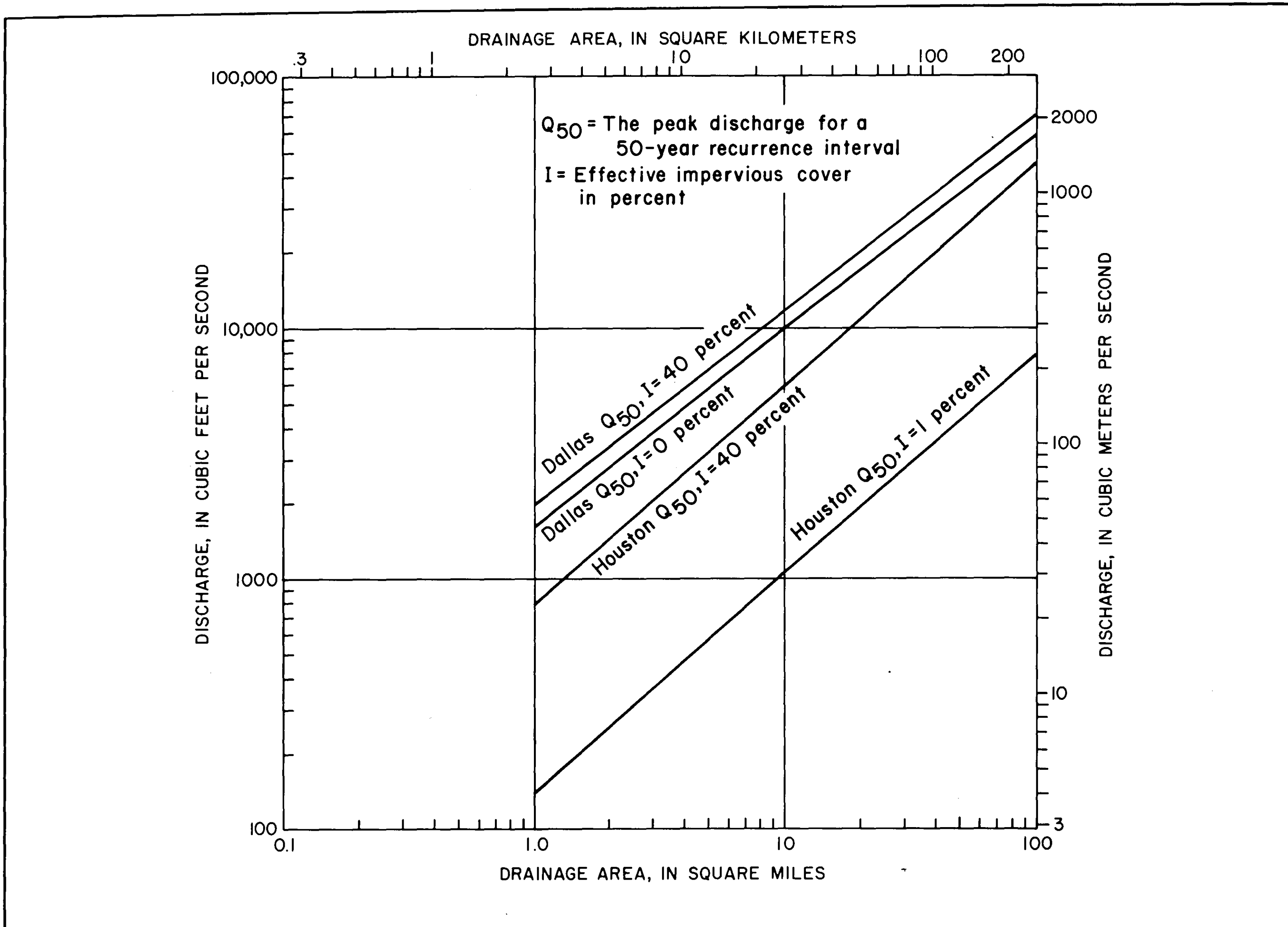


FIGURE 9.-Comparison showing the effects of urbanization on the 50-year peak discharges in Dallas and Houston

4. Low on-channel dams and yard terracing effectively reduce the peak discharges.

5. Structures built across channels tend to retard large flows.

6. After urbanization, topsoil is introduced into residential areas, thus increasing the depth of soils.

7. Excavation of rock for septic tanks and foundations tend to increase the infiltration rate.

Changes in the Volume of Annual Runoff

Continuous runoff data are available since 1962 at four gaging stations in three basins--Bachman Branch, Turtle Creek, and White Rock Creek basins (fig. 3). White Rock Creek Basin above Keller Springs Road is a rural basin; the intervening area between Keller Springs Road and Greenville Avenue is mostly urbanized. Bachman Branch Basin and Turtle Creek Basin are urban basins.

To estimate the approximate increase in annual runoff due to urbanization, the total and estimated direct runoff at the four gaging stations were compared. To determine the volume of annual direct runoff, hydrographs of average daily discharge for each water year were used.

The daily values of base flow were subtracted from the corresponding values of total flow for the estimated periods of direct runoff, and these differences were summed to obtain the annual volume of direct runoff. To account for the differences in drainage areas and for rainfall variability, the annual volumes of total and direct runoff were expressed as a percentage of rainfall that was computed by using appropriate rain-gage combinations and weighting methods.

Annual rainfall and runoff data for the four gaging stations and from the intervening part of White Rock Creek Basin between Keller Springs Road and Greenville Avenue are given in table 11. The data show that the average base flow is about 7 percent of the total runoff. The data also show that the urbanized basins have larger percentages of runoff.

To determine the approximate increase in runoff due to urbanization, the ratios of direct runoff in urban areas to direct runoff in rural areas were computed (table 11) for 1962-70. Figure 10 shows these ratios for the different basins, with the rural-basin ratio equal to 1.00 plotted against the percentage of effective impervious area in each basin (table 2).

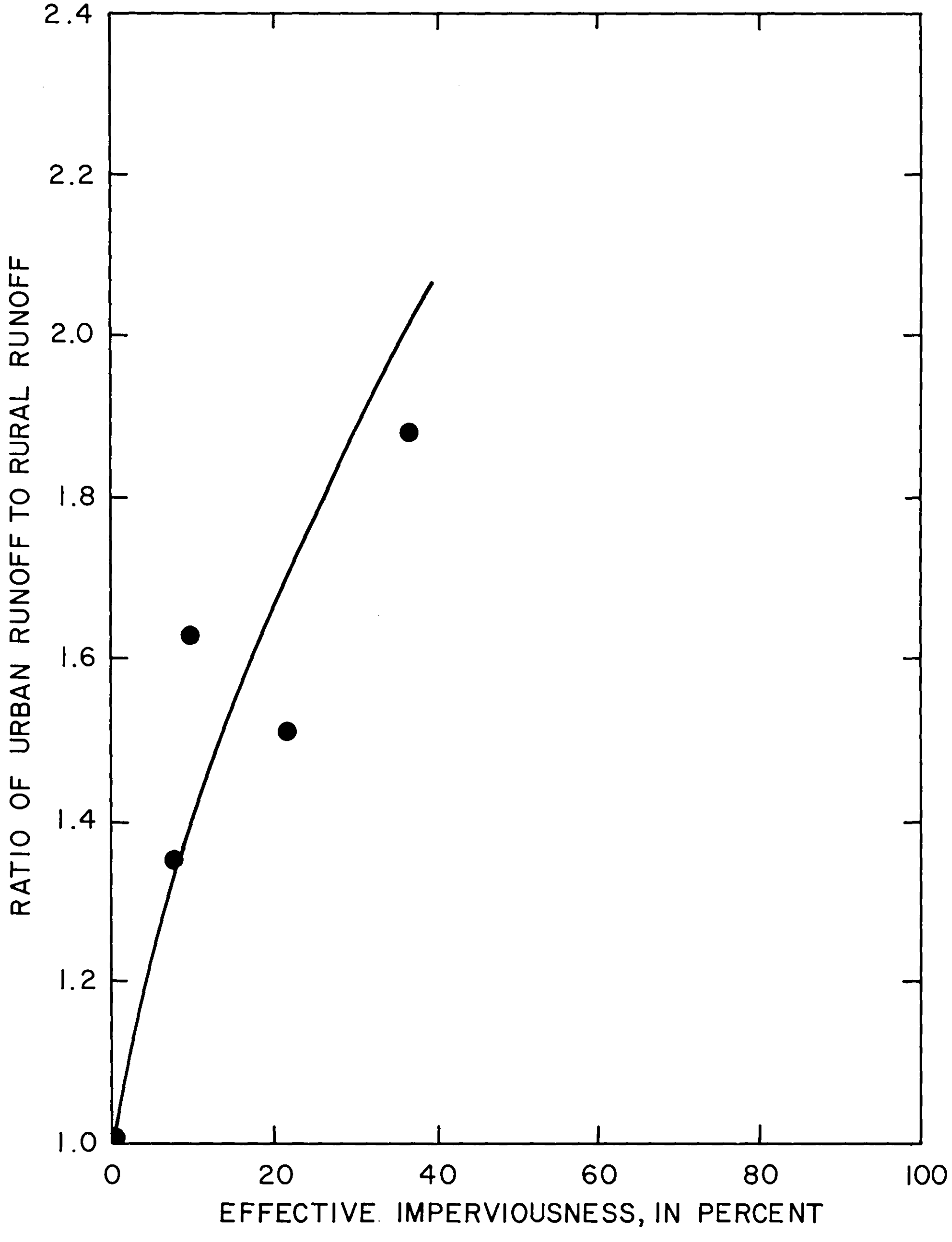


FIGURE 10.- Relation of effective impervious area to the ratio of urban runoff to rural runoff

Table 11.--Annual rainfall and runoff in selected basins 1962-70

Water year	Precipitation (inches)	Total runoff (percent of precipitation)	Direct runoff (percent of precipitation)	Precipitation (inches)	Total runoff (percent of precipitation)	Direct runoff (percent of precipitation)	Precipitation (inches)	Direct runoff (percent of precipitation)
				White Rock Creek				
	above Keller Springs Road (No. 8-571)			above Greenville Avenue (No. 8-572)			between Keller Springs Road and Greenville Avenue	
1962	43.75	22.7	18.2	45.11	30.0	24.0	45.56	28.8
1963	22.72	18.7	9.7	25.93	26.8	16.1	27.00	21.3
1964	43.61	30.6	30.2	41.77	30.7	28.1	41.16	25.8
1965	36.45	31.5	21.8	37.74	40.2	31.5	38.18	39.3
1966	42.97	28.3	25.0	46.42	39.1	34.5	47.55	42.4
1967	26.13	10.0	6.9	28.04	16.5	13.4	28.71	18.4
1968	35.99	25.4	18.2	38.53	32.9	22.7	39.38	26.6
1969	35.96	25.5	18.8	34.81	35.5	27.5	34.46	34.3
1970	43.66	18.6	13.1	41.87	30.2	20.5	41.27	26.4
Average	36.80	23.5	18.0	37.80	31.3	24.3	38.14	29.3
Ratio	-	-	1.00	-	-	1.35	-	1.63
	Bachman Branch (No. 8-557)			Turtle Creek (No. 8-565)				
1962	-	-	-	43.75	38.7	30.7		
1963	-	-	-	28.46	41.6	33.6		
1964	34.57	28.8	24.1	32.03	44.6	37.7		
1965	36.00	38.5	27.5	38.79	45.0	36.6		
1966	44.50	45.6	39.3	46.82	49.7	43.4		
1967	26.13	14.4	10.2	24.62	31.7	23.6		
1968	39.97	40.0	31.4	40.64	39.3	29.0		
1969	36.08	35.3	29.5	36.28	44.4	36.2		
1970	44.37	35.6	28.4	43.36	42.6	34.2		
Average	37.37	34.0	27.2	37.19	42.0	33.9		
Ratio	-	-	1.51	-	-	1.88		

The trend of the data points on figure 10 show that the runoff ratios increase with increasing impervious area, indicating that urbanization increases the volume of direct runoff. The curve shown on figure 10 was fitted to the data by the method of least squares.

Thirty-seven percent effective impervious area (Turtle Creek Basin) is about the maximum for fully developed residential basins in Dallas. At 37 percent effective impervious area, average annual direct runoff is about double that of an undeveloped area. Sufficient data have not been collected in a highly industrialized area, where the effective impervious area approaches 100 percent, to determine the increase in runoff.

SUMMARY AND CONCLUSIONS

The collection of streamflow and rainfall data in and near Dallas during the period 1962-70 has afforded a definition of some of the hydrologic effects attributable to urban development. A digital model of streamflow response to rainfall and evaporation input was calibrated for watersheds with different degrees of urban development as reflected by impervious area.

The rainfall-runoff relations were used with a 57-year record of rainfall to simulate annual peak discharges at 14 sites. Frequency curves were then prepared from these peak discharges, and from these, the discharges corresponding to recurrence intervals of 1.25, 2, 5, 10, 25,50, and 100 years were obtained.

The discharges at these recurrence intervals were related to drainage area, length-root slope ratio, and percentage of impervious area by multiple-regression techniques. These regional relations provide a method of estimating the flood-peak characteristics at ungaged sites. Based on these analyses, changing a rural basin to a fully developed residential urban basin will increase the flood peak at the 2-year recurrence interval by about 1.4 times, at the 10-year recurrence interval by about 1.2 times, and at the 50-year recurrence interval by about 1.2 times.

The data indicated that runoff in a fully developed residential area of about 40 percent effective impervious cover would be about double that of an undeveloped area.

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