ESTIMATING PEAK DISCHARGES, FLOOD VOLUMES, AND HYDROGRAPH SHAPES OF SMALL UNGAGED URBAN STREAMS IN OHIO

by James M. Sherwood

U.S. GEOLOGICAL SURVEY

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Final report on a study of the

EFFECTS OF URBANIZATION ON RUNOFF

FROM SMALL DRAINAGE AREAS IN OHIO,

prepared in cooperation with the

OHIO DEPARTMENT OF TRANSPORTATION

and the FEDERAL HIGHWAY ADMINISTRATION

Columbus, Ohio

1986

DEPARTMENT OF THE INTERIOR

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CONVERSION FACTORS

For the convenience of readers who may prefer to use metric (International System) units rather than the inch-pound units used in this report, values may be converted by using the following factors:

Multiply inch-pound unit	By	To obtain metric units
inch (in.)	25.40	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
foot per mile (ft/mi)	0.1894	meter per kilometer (m/km)
square inch ($in.^2$)	6.452	square centimeter (cm^2)
square mile (mi^2)	2.590	square kilometer (km^2)
cubic foot (ft^3)	0.02832	cubic meter (m^3)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)

<u>National Geodetic Vertical Datum of 1929 (NGVD of 1929):</u> A geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called "<u>Mean Sea Level</u>."

GLOSSARY

A	Drainage area (in square miles)--The contributing drainage area of a stream at a specified location, measured in a horizontal plane and enclosed by a topographic divide. It usually is computed from U.S. Geological Survey 7.5-minute topographic quadrangle maps. In urban areas, sewer maps also may be necessary because sewer lines frequently cross topographic divides.
BDF	Basin-development factor--A measure of channel and basin development that accounts for channel improvements, impervious channel linings, storm sewers, and curb-and-gutter streets. It is measured on a scale from 0 (little or no development) to 12 (fully developed). See pages 33-8 of this report for a more complete description and method of computation.
$\overline{C_s}$	Average coefficient of skew--Computed in this study by averaging the skew coefficients of the logarithms of the synthetic annual peak discharges of the 30 study sites.
EL	Average basin elevation index (in thousands of feet above NGVD of 1929)--Computed by averaging the elevations at the 10- and 85-percent distance points along the main channel, as determined from topographic maps.
$\overline{I_V}$	The average index of variability--Computed in this study by averaging the standard deviations of the logarithms of the synthetic annual peak discharges of the 30 study sites.
IA	Impervious area (in percent)--That part of the drainage area covered by impervious surfaces such as streets, parking lots, buildings, and so forth.
K	Coefficient of imperviousness--Computed as $K = 1+0.015IA$ (Carter, 1961), where IA is impervious area.
L	Main-channel length (in miles)--Distance measured along the main channel from the ungaged site to the basin divide, as determined from topographic maps.
LT	Basin lagtime (in hours)--Generally defined as the time elapsed from the centroid of the rainfall excess to the centroid of the resultant runoff hydrograph. Lagtime for the 30 basins in this study was computed as KSW + 1/2 TC--a relation previously defined by Kraijenhoff van de Leur (1966)--where KSW (linear reservoir-routing coefficient) and TC (time of concentration) are parameter values computed in the final model calibrations for each site.

Q_p Peak discharge (in cubic feet per second)--The maximum discharge (streamflow) of a flood event.

Q_t Discharge (in cubic feet per second)--Estimated discharge (streamflow) at time t.

rho The average interstation correlation coefficient--Computed in this study by averaging the correlation coefficients of the synthetic annual peak discharges for a 10-year period (1960-69) for the 30 sites used in the regression analysis.

RI2-30 2-year, 30-minute rainfall (in inches)--The maximum 30-minute rainfall having a recurrence interval of 2 years, as determined from U.S. Weather Bureau Technical Paper 40 (U.S. Department of Commerce, 1961).

RI2-120 2-year, 120-minute rainfall (in inches)--The maximum 120-minute rainfall having a recurrence interval of 2 years, as determined from U.S. Weather Bureau Technical Paper 40 (U.S. Department of Commerce, 1961).

RMS Root mean squared error (in logarithmic units and percent)--Used in the error analysis of the nationwide equations (Sauer and others, 1983) and computed as:

$$RMS = \sqrt{\bar{x}^2 + s^2}$$

RQ_x Rural peak discharge (in cubic feet per second)--The estimated rural peak discharge with recurrence of x years, as computed from regionalized regression equations developed by Webber and Bartlett (1977).

s The average standard deviation (in logarithmic units and percent)--In this study, the average standard deviation of the errors between synthetic and estimated peak discharges that results from applying the nationwide equations to the Ohio data.

SEP Standard error of prediction (in percent)--An approximation of the ability of a regression equation to estimate the peak discharge for a given recurrence interval at a site not used in the regression analysis. In this study, it was computed using a method described by Hardison (1971).

SER Standard error of regression (in percent)--A measure of the ability of a regression equation to estimate the peak discharge for a given recurrence interval at a gaged site used in the regression analysis.

SL Main-channel slope (in feet per mile)--Computed as the difference between the elevations (in feet) at 10 and 85 percent of the main-channel distance from the ungaged site to the basin divide, divided by the channel distance (in miles) between the two points, as determined from topographic maps.

ST Basin storage (in percent)--That part of the drainage area occupied by lakes, reservoirs, ponds, streams, and swamps, as determined from topographic maps. Temporary in-channel storage as a result of ponding behind roadway embankments and detention basins is not included.

t Time (in hours)--Associated with the estimated discharge Q_t.

UQ_x Urban peak discharge (in cubic feet per second)--The synthesized or estimated urban peak discharge with recurrence interval of x years; computed from flood-frequency analysis of synthetic long-term annual peak discharge data, or estimated from the regression equations presented in this report.

unit day Day for which data (rainfall or discharge) was collected at a 5-minute record interval.

V Flood volume (in cubic feet)--The total volume of direct runoff for a flood event; computed by integrating the area under the flood-event hydrograph, or estimated from the regression equation presented in this report.

$\bar{X}$ Mean error (in logarithmic units and percent)--In this study, the mean error was computed by subtracting the synthetic annual peak discharges from the annual peak discharges that were estimated by applying the nationwide equations (Sauer and others, 1983) to the Ohio data and averaging the differences.

ESTIMATING PEAK DISCHARGES, FLOOD VOLUMES, AND HYDROGRAPH SHAPES OF SMALL UNGAGED URBAN STREAMS IN OHIO

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ABSTRACT

Methods are presented for estimating peak discharges, flood volumes and hydrograph shapes of small (less than 5 square miles) urban streams in Ohio. Examples of how to use the various regression equations and estimating techniques also are presented.

Multiple-regression equations were developed for estimating peak discharges having recurrence intervals of 2, 5, 10, 25, 50, and 100 years. The significant independent variables affecting peak discharge are drainage area, main-channel slope, average basin-elevation index, and basin-development factor. Standard errors of regression and prediction for the peak discharge equations range from ±37 percent to ±41 percent.

An equation also was developed to estimate the flood volume of a given peak discharge. Peak discharge, drainage area, main-channel slope, and basin-development factor were found to be the significant independent variables affecting flood volumes for given peak discharges. The standard error of regression for the volume equation is ±52 percent.

A technique is described for estimating the shape of a runoff hydrograph by applying a specific peak discharge and the estimated lagtime to a dimensionless hydrograph. An equation for estimating the lagtime of a basin was developed. Two variables--main-channel length divided by the square root of the main-channel slope ($L/\sqrt{SL}$) and basin-development factor--have a significant effect on basin lagtime. The standard error of regression for the lagtime equation is ±48 percent.

The data base for the study was established by collecting rainfall-runoff data at 30 basins distributed throughout several metropolitan areas of Ohio. Five to eight years of data were collected at a 5-minute record interval. The U.S. Geological Survey rainfall-runoff model A634 was calibrated for each site. The calibrated models were used in conjunction with long-term rainfall records to generate a long-term streamflow record for each site. Each annual peak-discharge record was fitted to a Log-Pearson Type III frequency curve. Multiple-regression techniques were then used to analyze the peak discharge data as a function of the basin characteristics of the 30 sites.

INTRODUCTION

Urban development in Ohio has increased dramatically in recent years. Associated increases in impervious areas, storm-sewer developments, and stream-channel improvements have caused significant increases in the magnitudes of annual peak discharges. Highway and bridge engineers require reliable estimates of flood characteristics of streams to safely and efficiently design riverine structures such as bridges and culverts. Previous publications (Cross, 1946; Cross and Webber, 1959; Cross and Mayo, 1969; and Webber and Bartlett, 1977) have outlined techniques for estimating the flood-frequency relation of streams in rural areas of Ohio. To date, no methods have been published for estimating flood characteristics of streams in urban and suburban areas of Ohio.

In 1974, the U.S. Geological Survey, in cooperation with the Ohio Department of Transportation and the Federal Highway Administration, initiated an 11-year study of 30 partly to fully urbanized basins in Ohio (fig. 1, table 1). The objectives of this study titled "Effects of Urbanization on Runoff From Small Drainage Areas in Ohio" were to:

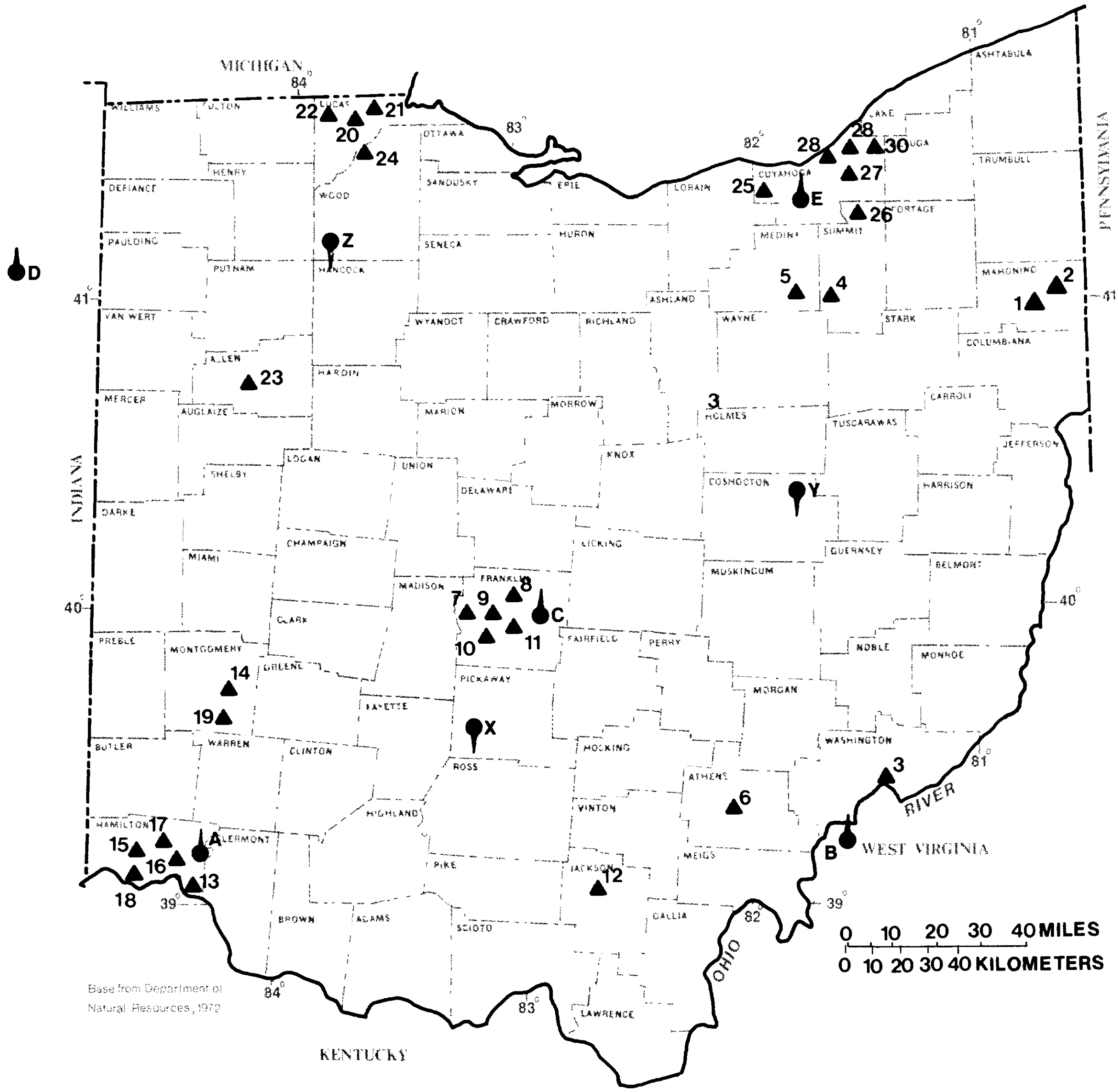
1. Establish a data base of magnitudes and frequencies of annual peak discharges for 30 basins distributed throughout the metropolitan areas of Ohio.

2. Develop statewide regression equations for estimating magnitudes and frequencies of annual peak discharges at ungaged urban sites from physical and climatic variables and a measure of basin development.

3. Develop methods for estimating flood volumes and hydrograph shapes at ungaged sites.

Purpose and Scope

The purpose of this report is to summarize the methods of data collection and analysis for this study; to present the regression equations developed for estimating peak discharges, flood volumes, and basin lagtimes; and to present a method for estimating hydrograph shapes. Step-by-step examples of how to use the equations and how to estimate hydrograph shapes also are presented. The equations and methods developed are applicable to small urban basins in Ohio whose basin characteristics are within the ranges of the basin characteristics of the 30 study sites: Drainage area, 0.026-4.09 square miles (mi^2); main-channel slope, 8.00-462 feet per mile (ft/mi); average basin-elevation index, 0.622-1.21 thousand feet; and basin-development factor on a scale of 0 to 12.



EXPLANATION

▲21 U.S. Geological Survey rainfall-runoff station

C National Weather Service long-term rainfall station

X National Weather Service evaporation station

Figure 1.--Approximate locations of rainfall-runoff stations, long-term rainfall stations, and evaporation stations. (See tables 1, 4, and 5 for cross-reference of map symbols.)

Table 1.--Map location numbers, latitudes, and longitudes of study sites

[dec. deg., decimal degrees]

Station number	Station name	Map location number	Latitude (dec. deg.)	Longitude (dec. deg.)
03098350	Charles Ditch at Boardman	1	41.0119	80.6622
03098900	Bunn Brook at Struthers	2	41.0514	80.6078
03115810	Rand Run at Marietta	3	39.4133	81.4289
03115995	Sweet Henri Ditch at Norton	4	41.0253	81.6419
03116150	Orchard Run at Wadsworth	5	41.0386	81.7328
03159503	Home Ditch at Athens	6	39.3350	82.0786
03221450	Fishinger Creek at Upper Arlington	7	40.0356	83.0786
03226860	Rush Run at Worthington	8	40.0947	82.9989
03226900	Fishinger Road Creek at Upper Arlington	9	40.0236	83.0444
03227050	Norman Ditch at Columbus	10	39.9930	83.0478
03228950	Dawnlight Ditch at Columbus	11	40.0142	82.9461
03236050	Coalton Ditch at Coalton	12	39.1056	82.6130
03238790	Anderson Ditch at Cincinnati	13	39.0736	84.3833
03241850	Gentile Ditch at Kettering	14	39.7130	84.1489
03256250	Springfield Ditch near Cincinnati	15	39.2192	84.5192
03258520	Amberly Ditch near Cincinnati	16	39.1869	84.4294
03259050	Wyoming Ditch at Wyoming	17	39.2364	84.4936
03260095	Delhi Ditch near Cincinnati	18	39.1033	84.6167
03271295	Whipps Ditch near Centerville	19	39.6436	84.1508
04176870	Ketchum Ditch at Toledo	20	41.7108	83.5958
04176880	Silver Creek at Toledo	21	41.7200	83.6389
04176890	Tifft Ditch at Toledo	22	41.7033	83.6542
04187700	Pike Run at Lima	23	40.7703	84.1069
04193900	Grassy Creek at Perrysburg	24	41.5633	83.6172
04200800	Glen Park Creek at Bay Village	25	41.4850	81.9214
04207110	Tinkers Creek Tributary at Twinsburg	26	41.3250	81.4797
04208580	N. F. Doan Brook at Shaker Heights	27	41.4825	81.5417
04208640	Dugway Brook at Cleveland Heights	28	41.4972	81.5369
04208680	Euclid Creek Tributary at Lyndhurst	29	41.5208	81.4889
04208685	Mall Run at Richmond Heights	30	41.5430	81.4983

Acknowledgments

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Special recognition and appreciation are also extended to: William P. Bartlett, Jr., who was project chief from 1978-84; Don D. Brooks, for his steadfast dedication to data collection; and G. F. Koltun, for his assistance with the statistical analyses.

DATA COLLECTION

Rainfall and runoff data were collected at 5-minute intervals at 30 small urban basins (drainage area less than 5 square miles) for periods ranging from 5 to 8 years (fig. 1, table 1). Sites were chosen in which no change in the level of urban development was anticipated for the study period.

The stream gage at each site consisted of a digital recorder housed in a 20-inch, cube-shaped steel shelter. The shelter was mounted on a 3-inch aluminum stilling well positioned at the upstream end of the culvert. The well intake was a 2-inch-diameter steel pipe with a static tube on the end. The intake extended from the base of the well to a point one culvert width upstream from the culvert entrance. A crest-stage gage was mounted at the downstream end of the culvert to verify the non-occurrence of type 3 or type 4 flow (backwater conditions requiring measurement of stage downstream). Downstream stage recorders were necessary at 5 of the 30 sites because of the occurrence of type 3 flow. Stage-discharge relationships were developed for each site using procedures outlined by Bodhaine (1968).

Rainfall data were recorded by another digital recorder housed in a similar steel shelter with a 50-square-inch rainfall collector on top. The shelter was mounted on a 3-inch aluminum float well. A tube inside the shelter connected the collector to the float well. The rain gage was installed at the stream-gage site if the rainfall would not be obstructed by surrounding trees. Otherwise, the rain gage was installed at an unobstructed, accessible location elsewhere within the basin. (A photograph of a typical rainfall-runoff data-collection station is shown in figure 2.) If periods of daily rainfall were missing due to a faulty recorder or the winter shutdown period, data from a nearby rainfall station operated by the National Weather Service were substituted.

Daily rainfall totals were stored for all days, and 5-minute unit data for rainfall and runoff were stored for all rainfall-runoff flood events.

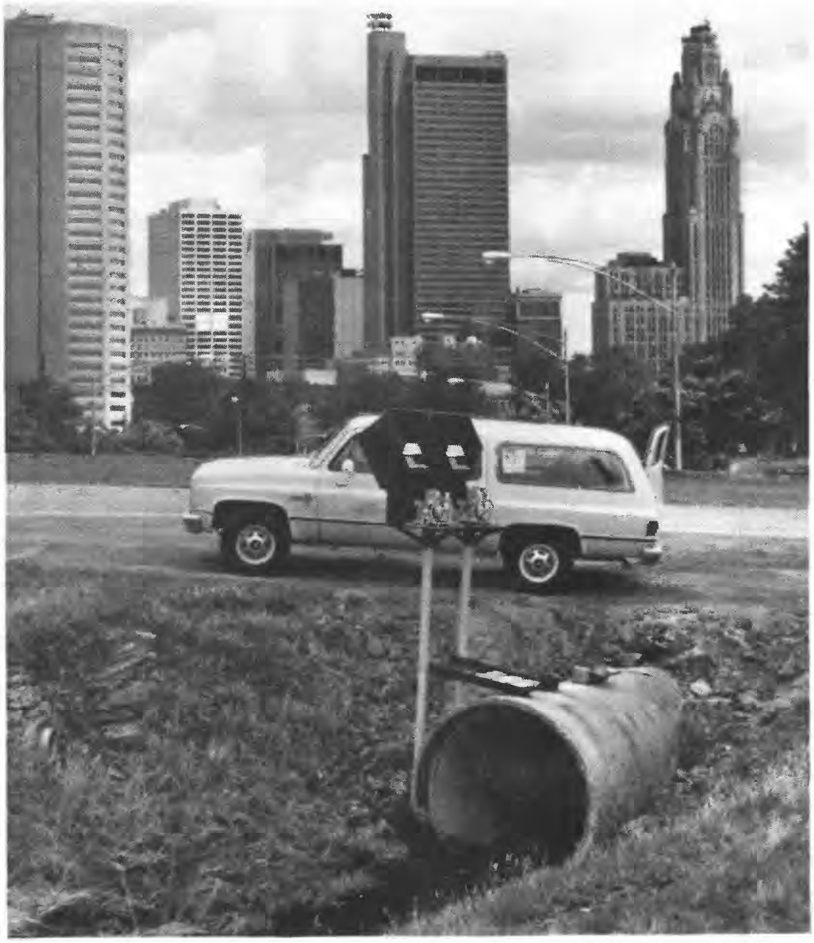


Figure 2.--Typical rainfall-runoff data-collection station.

Data collection was discontinued during the winter months (mid-December to mid-March) because (1) the instruments were not capable of recording snow accumulations, (2) the rainfall-runoff model is not capable of simulating snowmelt runoff, and (3) the stage-discharge relations were valid only for unobstructed, ice-free culvert flow. Because the major thunderstorm periods that produce annual peak discharges on small streams occur during the spring and summer months, the loss of usable rainfall-runoff data resulting from the winter shutdown is minimal.

Additional data required for model calibration and synthesis are daily pan evaporation, long-term unit (5-minute) rainfall for selected storm periods, and long-term daily rainfall. These data were obtained from eight National Weather Service stations (fig. 1).

All data were stored in the U.S. Geological Survey's WATSTORE system (National Water Data Storage and Retrieval System) (Hutchinson, 1975).

DATA ANALYSIS

The following sections on model calibration and long-term synthesis refer to and briefly describe several computer programs. Documentation on the operation of these programs is contained in a user's guide by Carrigan, Dempster, and Bower (1977).

Calibration of the Rainfall-Runoff Model

Calibrated rainfall-runoff models are frequently used to synthesize long-term runoff records from long-term rainfall records. Synthesis of record significantly shortens the data-collection period required for flood-frequency analysis. The technique is particularly well suited to urban studies for which a shorter data-collection period can minimize the effect of increased urbanization within the period.

The U.S. Geological Survey rainfall-runoff model (computer program A634) used for this study was originally developed by Dawdy, Lichty, and Bergman (1972) and was refined by Carrigan (1973), Boning (1974), and Carrigan, Dempster, and Bower (1977). Model A634 was selected over other rainfall-runoff models because it is reliable and is the least costly and time consuming in terms of data collection and model calibration. Input data required for model calibration are daily rainfall, daily evaporation, unit rainfall, and unit discharge. The ten parameters within the model interact to simulate the hydrologic processes of antecedent soil moisture, infiltration, and surface-runoff routing (table 2). The antecedent soil-moisture accounting component contains four parameters (EVC, RR, BMSM, DRN) and uses daily rainfall and daily

Table 2.--Rainfall-runoff model parameters

[Dash in units column indicates dimensionless parameter]

Parameter	Units	Definition
Antecedent soil-moisture accounting component		
BMSM	inches	Soil moisture storage volume at field capacity.
EVC	--	Coefficient to convert pan evaporation to potential evapotranspiration.
RR	--	Proportion of daily rainfall that infiltrates the soil.
DRN	inches per hour	The constant rate of drainage for redistribution of soil moisture.
Infiltration component		
PSP	inches	Minimum value of the combined action of capillary suction and soil moisture differential.
KSAT	inches per hour	Minimum saturated hydraulic conductivity used to determine soil infiltration rates.
RGF	--	Ratio of combined action of suction and potential at wilting point to that at field capacity.
Surface-runoff routing component		
KSW	hours	Linear reservoir routing coefficient.
TC	minutes	Duration of the triangular translation hydrograph (time of concentration).
TP/TC	--	Ratio of time to peak to time of concentration.

evaporation data to simulate the redistribution of moisture in the soil column and evapotranspiration from the soil. The infiltration component contains three parameters (PSP, KSAT, RGF) and uses unit (5-minute) rainfall data and the results from the soil-moisture computations to compute rainfall excess (rainfall minus infiltration). The surface-runoff routing component contains three parameters (KSW, TC, TP/TC) and uses the Clark unit-hydrograph method to transform the rainfall excess into the outflow hydrograph. The ten parameters were optimized by a trial-and-error, hill-climbing technique based on a method devised by Rosenbrock (1960).

Each site was calibrated in three phases. During the first phase, the parameters controlling the volume of the simulated hydrograph (PSP, KSAT, DRN, RGF, BMSM, EVC, RR) were optimized while the values of the parameters controlling hydrograph shape (KSW, TC, TP/TC) were fixed. In phase two, the shape parameters were optimized while the volume parameters were fixed. In phase three, the parameters optimized in phase one were readjusted for a best fit of simulated peaks to observed peaks.

After an initial calibration, selected events were excluded from further calibrations on the basis of the following criteria:

1. A well-distributed, broad range of peak discharges is desirable for accurate model calibration. Peaks of record for the 30 sites ranged from floods of 2-year to 100-year recurrence interval, with the median being a 4-year flood. As might be expected, the data sets for many of the 30 sites contained a disproportionately large number of very small events. Because the calibrated models were to be used to synthesize relatively large events (annual peak discharges), most of the very small events were excluded from model calibration. This was accomplished by excluding all events below a specified minumum peak discharge--for example, 4.0 ft^3/s (cubic feet per second).

2. Because uniform distribution of rainfall over the basin is a major assumption of the model, any events with obviously unrepresentative rainfall (such as total rainfall less than total runoff) were excluded.

3. Events were excluded if field notes indicated that the culvert entrance may have been partially obstructed during the event, which could result in an exaggerated observed hydrograph.

4. If obvious data problems (such as snowmelt, plugged rainfall collector, recorder malfunction) occurred, these events were excluded.

One or more additional calibrations were made until a reasonable fit of simulated versus observed peaks was achieved.

Figure 3 is an example of a computer-generated plot of the observed hydrograph, simulated hydrograph, and hyetograph (plot of rainfall intensity versus time) for a single event. A plot of this type is generated for each event (including those events excluded from model calibration). The plot assists the modeler in adjusting beginning and ending times for each flood event, identifying unrepresentative rainfall or obvious data problems, and evaluating how well the model is simulating direct runoff.

Figure 4 compares simulated flood peaks to observed flood peaks. A plot of this type is generated at the end of each calibration run and assists the modeler in evaluating data distribution, identifying events that may have unrepresentative rainfall or obvious data problems, identifying any apparent bias within the range of peak discharges, and evaluating how well the model is simulating flood peaks.

Table 3 summarizes calibration results. Average standard errors of estimate (in percent) are presented for peak discharges at each site.

Peak-Discharge Synthesis and Frequency

Peak discharges for each basin were synthesized with the U.S. Geological Survey synthesis model (computer program E784). The model uses the final parameter values from the calibrated rainfall-runoff model in combination with long-term rainfall and evaporation records to produce a long-term record of synthetic peak discharges. Data from the closest long-term rainfall and evaporation stations were used to synthesize the long-term peak discharge records for each of the 30 sites as indicated in table 3.

Data were available from five long-term rainfall stations operated by the National Weather Service (fig. 1). Computer program G159 was used to select the unit rainfall data to be used in the long-term synthesis. This was accomplished by scanning the daily rainfall records and selecting up to five of the highest events for each year. An average of three events per year were selected. The daily totals and selected unit days are the rainfall input for the model. The periods of record and number of unit days for each of the five stations are summarized in table 4.

Data were available from three evaporation data stations operated by the National Weather Service (fig. 1). Ten years of observed record at each site were used to generate an 85-year synthetic record using computer program H266. The program fits a harmonic (sine-cosine) function to the observed daily data by least squares. Daily evaporation data used in the synthesis model are summarized in table 5.

A flood-frequency curve was defined for each site by fitting a Log-Pearson Type III distribution to the annual peak discharges as recommended by the Water Resources Council (1981). The skew coefficient used for each site was computed directly from the synthesized data. The regional map skew provided by the Water Resources Council (1981) was considered less reliable, as it was developed from rural data and does not represent urban conditions.

A second set of flood-frequency data was estimated for each site using regionalized regression equations developed by Webber and Bartlett (1977), which are applicable to rural basins in Ohio ranging in size from 0.01 to 7,400 square miles. This data set is an estimation of the flood-frequency relationship for the study basins prior to their urban development; the data were used for comparison purposes and as independent variables in the regression analysis. Both sets of flood-frequency data for the 30 sites are summarized in table 6.

It should be noted here that the computed rural peak discharges are higher at some sites than the synthesized urban peak discharges. The situation occurs more frequently at higher recurrence intervals, and may be a result of at least two factors:

1. Urbanization does not always increase peak discharges. Temporary in-channel storage behind underdesigned culverts and bridges can reduce peaks. Also, if only the lower part of a basin has been developed, the discharge from the lower part may leave the basin before the discharge from the upper part arrives downstream, thus reducing the peak. This results in a tendency for urban and rural flood-frequency curves to converge (or cross) at higher recurrence intervals (figure 5).

2. The combination of several sources of error may also be a factor: (a) The standard errors of regression of the equations used to estimate the rural peak discharges range from ±26 percent to ±41 percent, (b) the standard errors of estimate of the individual calibrated models used to synthesize the urban peak discharges range from ±14 to ±34 percent, and (c) errors in stage-discharge relationships are estimated at ±5-10 percent, and contribute to the errors in both the estimated rural peak discharges and synthesized urban peak discharges.

It also should be noted that the urban flood-frequency curves in figure 5 are flatter than the rural flood-frequency curves. This may result from possible loss of variance in the synthesized urban flood-frequency data. Previous investigations indicate that there tends to be less variability in synthesized flood-frequency estimates when compared to flood-frequency estimates based on observed data.

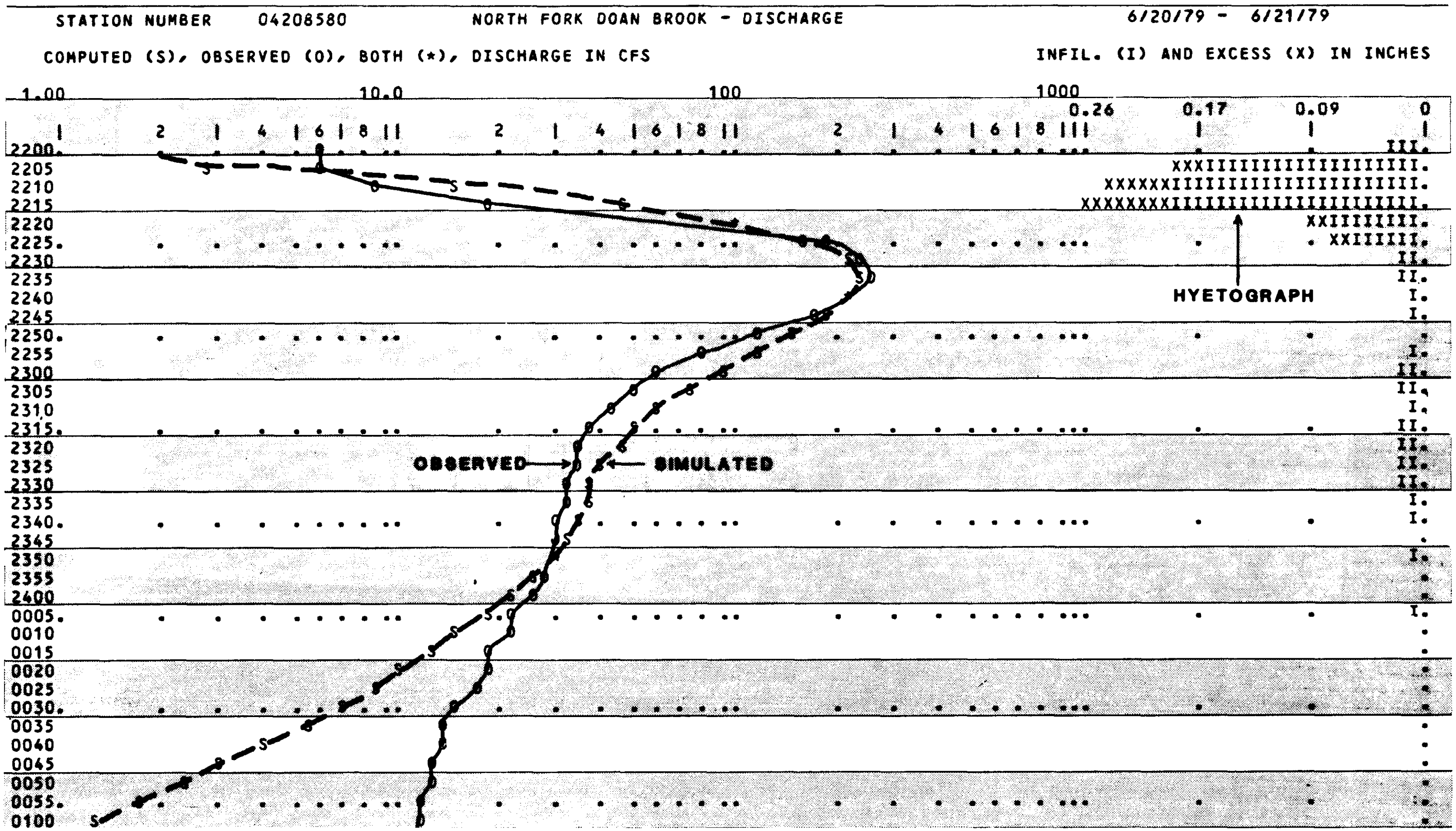
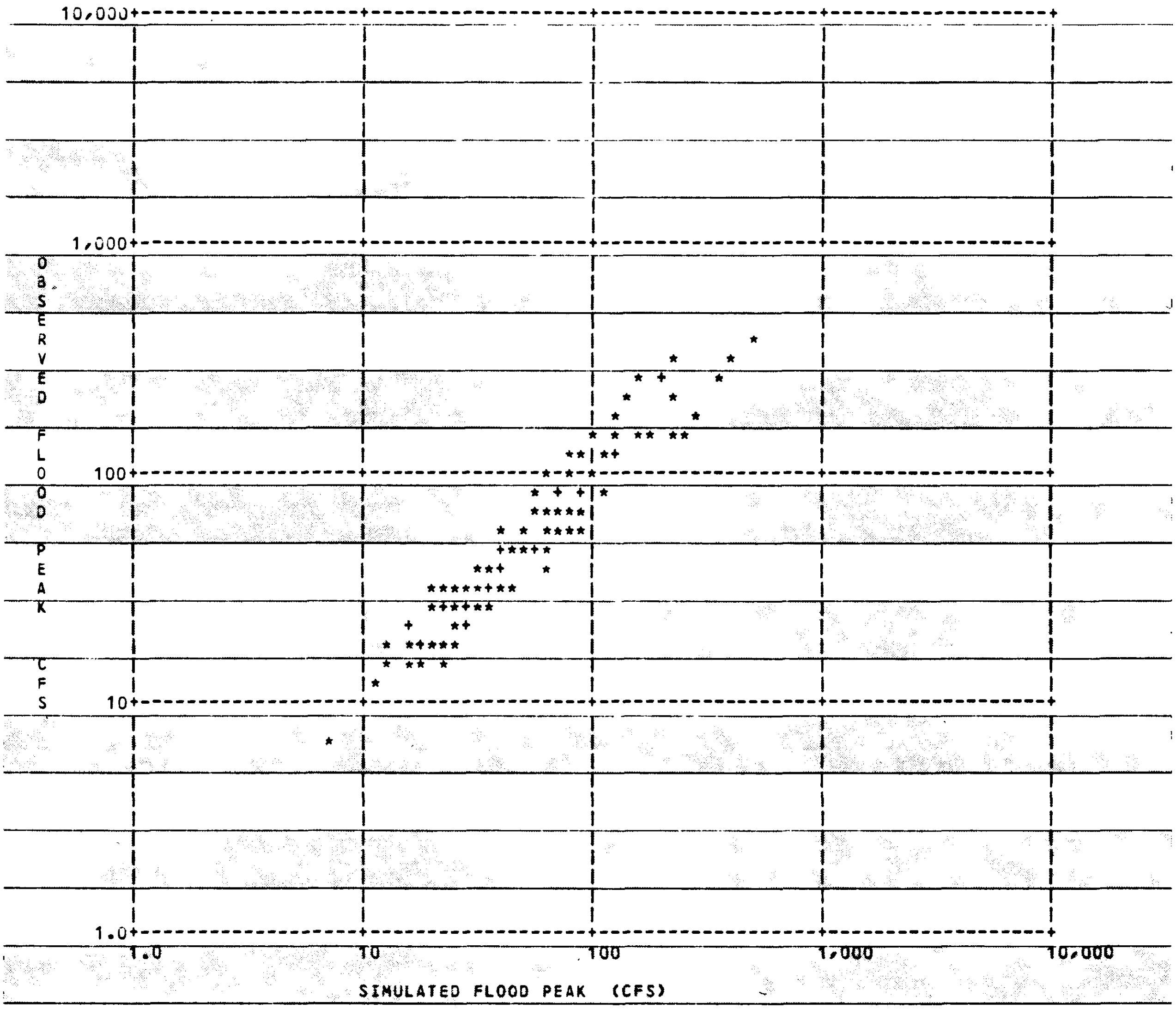


Figure 3.--Computer-generated graph of observed hydrograph, simulated hydrograph, and hyetograph.

USGS RAINFALL-RUNOFF MODEL CALIBRATION FOR 04208580 NORTH FORK DOAN BROOK - DISCHARGE



NUMBER OF POINTS PLOTTED (*) = 101
NUMBER OF MULTIPLE POINTS (+) = 19
NUMBER OF POINTS OFF GRID (X) = 0
NUMBER OF POINTS OMITTED = 26

*** NOTE ***
PLOT ROUTINE EXCLUDES
POINTS NOT INCLUDED
IN CALIBRATION

Figure 4.--Graph comparing observed flood peaks to simulated flood peaks.

Table 3.--Rainfall-runoff model and synthesis model information

Station number	Station name	Observed record: Number of years	Observed record: Period	Number of events[1]	Standard error of peak discharge (percent)	Map-location symbols of stations used for long-term synthesis: Rainfall	Map-location symbols of stations used for long-term synthesis: Evaporation
03098350	Charles Ditch at Boardman	7	1978-1984	94	17.1	E	Y
03098900	Bunn Brook at Struthers	5	1978-1982	55	17.3	E	Y
03115810	Rand Run at Marietta	7	1977-1983	60	24.5	B	Y
03115995	Sweet Henri Ditch at Norton	5	1978-1982	95	21.7	E	Y
03116150	Orchard Run at Wadsworth	6	1978-1983	66	23.1	E	Y
03159503	Home Ditch at Athens	8	1977-1984	60	24.4	B	X
03221450	Fishinger Creek at Upper Arlington	7	1975-1981	81	21.4	C	X
03226860	Rush Run at Worthington	8	1977-1984	38	27.8	C	X
03226900	Fishinger Road Creek at Upper Arlington	6	1975-1980	72	18.0	C	X
03227050	Norman Ditch at Columbus	6	1975-1980	55	17.3	C	X
03228950	Dawnlight Ditch at Columbus	7	1975-1981	66	26.3	C	X
03236050	Coalton Ditch at Coalton	7	1977-1983	34	28.9	B	X
03238790	Anderson Ditch at Cincinnati	7	1977-1983	37	17.2	A	X
03241850	Gentile Ditch at Kettering	7	1977-1983	116	18.1	A	X
03256250	Springfield Ditch near Cincinnati	6	1977-1982	61	14.9	A	X
03258520	Amberly Ditch near Cincinnati	8	1976-1983	30	24.9	A	X
03259050	Wyoming Ditch at Wyoming	7	1976-1984	32	29.9	A	X
03260095	Delhi Ditch near Cincinnati	8	1976-1983	61	15.3	A	X
03271295	Whipps Ditch near Centerville	7	1977-1983	62	23.5	A	X
04176870	Ketchum Ditch at Toledo	8	1977-1984	39	28.0	D	Z
04176880	Silver Creek at Toledo	8	1977-1984	36	24.7	D	Z
04176890	Tifft Ditch at Toledo	8	1977-1984	66	25.9	D	Z
04187700	Pike Run at Lima	7	1978-1984	58	27.7	D	Z
04193900	Grassy Creek at Perrysburg	6	1979-1984	30	33.7	D	Z
04200800	Glen Park Creek at Bay Village	6	1979-1984	45	29.8	E	Y
04207110	Tinkers Creek Tributary at Twinsburg	6	1978-1983	35	24.9	E	Y
04208580	N. F. Doan Brook at Shaker Heights	7	1978-1984	101	25.0	E	Y
04208640	Dugway Brook at Cleveland Heights	5	1978-1982	54	26.4	E	Y
04208680	Euclid Creek Tributary at Lyndhurst	5	1978-1982	38	29.0	E	Y
04208685	Mall Run at Richmond Heights	5	1978-1982	115	14.5	E	Y

[1]This is the number of events used in the final model calibration and corresponds to the standard error of peak discharge.

Table 4.--National Weather Service rainfall stations used in synthesis of peak-discharge data

Station number	Location and map symbol (fig. 1)	Record		Number of events
		Number of years	Period	
390900084310000	Cincinnati, Ohio (A)--	80	1897-1976	247
391600081340001	Parkersburg, West Virginia (B)---	77	1899-1975	218
400000082530001	Columbus, Ohio (C)----	81	1897-1977	236
410000085130000	Fort Wayne, Indiana (D)---------	67	1911-1977	305
412400081510000	Cleveland, Ohio (E)---	88	1890-1977	171

Table 5.--National Weather Service evaporation stations used in calibration of the rainfall-runoff model and synthesis of peak-discharge data

Station number	Location and map symbol (fig. 1)	Observed record		Synthetic record	
		Number of years	Period	Number of years	Period
393800083130000	Deer Creek Lake, Ohio (X)---------	10	1975-1984	85	1890-1974
402200081480000	Coshocton, Ohio (Y)---------	10	1975-1984	85	1890-1974
411300083460000	Hoytville, Ohio (Z)---------	10	1975-1984	85	1890-1974

Table 6.--Independent-variable values and flood-frequency data summary

[ft, feet; ft/mi, feet per mile; ft^3/s, cubic feet per second; mi^2, square miles]

Station number	Station name	Independent variables: A (mi^2)	BDF	SL (ft/mi)	EL (thousands of ft)	Peak discharge (urban/rural), in ft^3/s, for indicated recurrence interval, in years: 2	5	10	25	50	100
03098350	Charles Ditch at Boardman	0.499	11	31.5	1.08	169	278	349	435	496	555
						59	101	134	178	212	248
03098900	Bunn Brook at Struthers	.512	8	58.3	1.04	69	113	143	183	213	243
						67	116	154	206	247	292
03115810	Rand Run at Marietta	.330	4	141	.659	41	67	87	113	133	154
						53	116	173	262	341	435
03115995	Sweet Henri Ditch at Norton	.356	5	72.2	1.08	131	229	300	392	461	531
						52	91	122	164	197	234
03116150	Orchard Run at Wadsworth	.432	11	116	1.21	175	306	399	517	606	694
						66	115	155	210	254	302
03159503	Home Ditch at Athens	.238	3	68.3	.755	65	112	144	182	208	232
						35	68	95	136	170	210
03221450	Fishinger Creek at Upper Arlington	.660	9	61.5	.838	169	287	370	477	557	636
						105	192	256	349	419	495
03226860	Rush Run at Worthington	.715	2	8.00	.912	42	63	78	99	115	133
						60	106	140	187	222	260
03226900	Fishinger Road Creek at Upper Arlington	.450	11	73.7	.834	207	295	351	418	466	512
						79	144	194	266	320	380
03227050	Norman Ditch at Columbus	.600	10	46.3	.808	167	241	288	343	383	420
						92	166	222	302	362	427
03228950	Dawnlight Ditch at Columbus	.195	8	65.0	.815	66	102	126	156	178	199
						41	77	104	143	173	205
03236050	Coalton Ditch at Coalton	.501	0	110	.770	58	125	183	269	342	422
						86	158	213	290	351	414
03238790	Anderson Ditch at Cincinnati	.049	8	333	.772	41	66	81	98	109	120
						17	33	46	64	79	94
03241850	Gentile Ditch at Kettering	.064	12	44.4	.945	58	82	97	113	123	133
						18	35	47	65	78	93

Table 6.--Independent-variable values and flood-frequency data summary--Continued

Station number	Station name	Independent variables: A (mi^2)	BDF	SL (ft/mi)	EL (thousands of ft)	Peak discharge (urban/rural), in ft^3/s, for indicated recurrence interval, in years: 2	5	10	25	50	100
03256250	Springfield Ditch nr Cincinnati	.257	9	117	.792	190 54	292 101	356 137	433 188	487 228	538 270
03258520	Amberly Ditch nr Cincinnati	0.138	9	287	.732	37 35	64 67	84 91	111 125	131 153	151 182
03259050	Wyoming Ditch at Wyoming	.026	11	462	.786	25 11	36 22	43 30	50 42	55 52	59 63
03260095	Delhi Ditch nr Cincinnati	.160	10	127	.860	87 39	137 74	169 100	206 138	233 168	257 200
03271295	Whipps Ditch nr Centerville	2.64	9	58.9	.940	711 357	1160 634	1450 837	1790 1120	2040 1340	2260 1570
04176870	Ketchum Ditch at Toledo	.839	10	13.0	.622	93 21	132 39	158 52	192 70	218 83	244 96
04176880	Silver Creek at Toledo	4.09	6	14.8	.630	181 82	246 143	288 185	338 242	374 284	408 325
04176890	Tifft Ditch at Toledo	.854	8	19.4	.639	90 24	123 44	145 58	173 77	194 92	215 107
04187700	Pike Run at Lima	1.18	7	24.8	.856	179 87	284 131	366 163	483 200	580 228	686 256
04193900	Grassy Creek at Perrysburg	1.81	6	8.58	.631	150 35	238 64	299 85	377 114	436 136	494 157
04200800	Glen Park Creek at Bay Village	1.21	4	48.6	.655	141 126	307 213	446 281	649 373	817 445	996 524
04207110	Tinkers Creek Tributary at Twinsburg	.118	3	94.9	1.14	35 23	62 41	82 56	109 76	130 91	151 109
04208580	N. F. Doan Brook at Shaker Heights	1.18	10	86.3	1.09	258 138	511 236	703 315	960 421	1160 506	1360 601
04208640	Dugway Brook at Cleveland Heights	1.42	12	70.9	.980	366 155	679 263	908 348	1210 464	1430 557	1650 659
04208680	Euclid Creek Tributary at Lyndhurst	1.67	11	44.0	1.03	378 161	590 271	725 356	885 471	996 562	1100 661
04208685	Mall Run at Richmond Heights	.159	12	78.5	.990	128 28	198 50	239 67	286 91	316 110	343 130

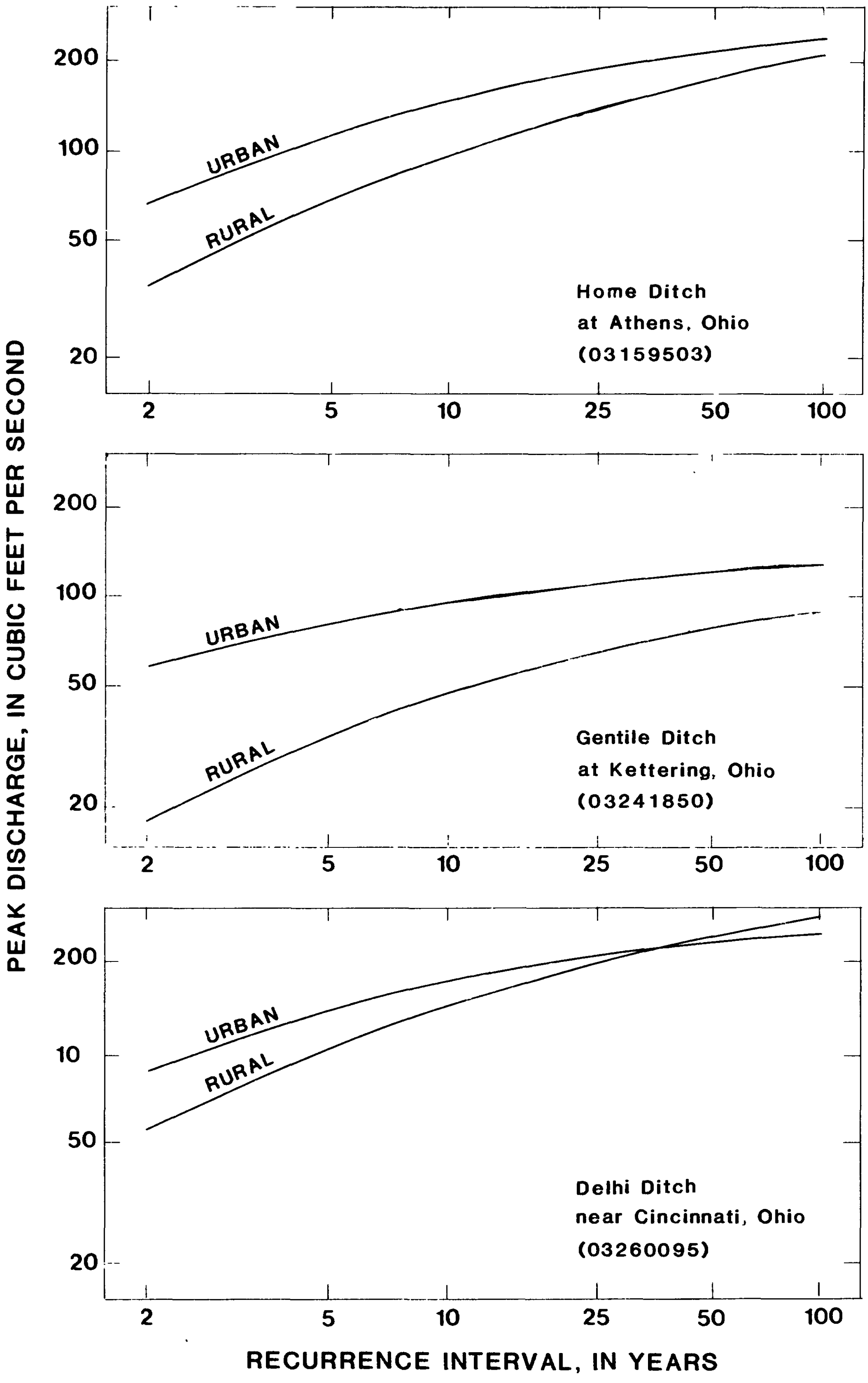


Figure 5.--Comparison of urban and rural flood-frequency curves.

Analysis of Peak Discharge as a Function of Basin Characteristics

Multiple regression techniques were used to develop equations for estimating the magnitudes of floods of selected recurrence intervals at ungaged urban sites. Peak discharges with recurrence intervals of 2, 5, 10, 25, 50, and 100 years were related to basin characteristics of the 30 sites using an equation of the general form:

$$UQ_x = a\, A^d\, B^e\, C^f \ldots ;$$

where:

UQ_x	is urban peak discharge with recurrence interval of x years (dependent variable),
a	is a regression constant,
A, B, C	are basin characteristics (independent variables), and
d, e, f	are regression coefficients.

Independent Variables Tested

The 13 basin characteristics initially tested in the regression analysis were:

RQ_x	--	equivalent rural peak discharge with recurrence interval of x years
BDF	--	basin-development factor
A	--	drainage area
IA	--	impervious area
K	--	coefficient of imperviousness
L	--	main-channel length
SL	--	main-channel slope
$L/\sqrt{SL}$	--	main-channel length, divided by the square root of the main-channel slope
LT	--	lagtime
EL	--	average basin-elevation index

P -- average annual precipitation

RI2-30 -- 2-year, 30-minute rainfall

RI2-120 -- 2-year, 120-minute rainfall.

The 13 variables (defined in the glossary) were chosen because of their significance in previous studies of similar purpose. Basin storage (ST) was not tested in the regression analysis because all sites were chosen to have little or no storage. (Of the 30 sites, 24 had no storage; of the six that had storage, the maximum was 0.20 percent.)

In a nationwide study of flood magnitudes and frequencies in urban areas, Sauer and others (1983) developed three sets of multiple-regression equations for estimating peak discharges at ungaged urban sites in the United States. The following independent variables were found to be significant.

7-parameter (preferred) equations	RQ_X, BDF, A, IA, SL, ST, RI2-120
3-parameter equations	RQ_X, BDF, A
7-parameter (alternate) equations	RQ_X, BDF, A, IA, SL, LT, RI2-120

Webber and Bartlett (1977) found A, SL, EL, P, and ST to be significant in equations for estimating rural peak discharges in Ohio.

$L/\sqrt{SL}$ also was tested, as it has been closely related to LT in several previous studies.

Equations Developed for Small Urban Streams in Ohio

The multiple-regression analysis was performed using the Statistical Analysis System[1] (SAS Institute, 1982). A combination of stepwise, step-forward, and step-backward procedures was used to determine which of the independent variables would be included in the six regression equations.

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[1]Use of trade names in this report is for identification purposes only and does not constitute endorsement by the U.S. Geological Survey.

The analysis resulted in six regression equations (table 7), which may be used to estimate the magnitudes of peak discharges for urban basins in Ohio within the accuracy and limitations discussed in subsequent parts of this report. The standard error of regression (SER) and standard error of prediction (SEP) also are given for each equation. The values of dependent (UQ_x) and independent (A, BDF, SL, EL) variables used in the regression analysis are presented in table 5.

In the nationwide study, Sauer and others (1983) found that if BDF was used on a reverse scale (13-BDF), the linearity of the equation was greatly improved and the standard error was reduced. In this study, both BDF and 13-BDF were tested; 13-BDF gave the best results.

The standard error of regression is an indication of how well the equations estimate peak discharges for the 30 gaged sites used in the regression analysis. The standard error of prediction, on the other hand, is an approximation of the ability of the regression equation to estimate the flood magnitude of a given recurrence interval at a site not included in the regression analysis. The method for computing the standard error of prediction is explained by Hardison (1971). Several factors are involved in the computation of SEP. Their values and methods of computation are as follows:

rho = 0.256	The average interstation correlation coefficient (rho) was computed by averaging the correlation coefficients of the synthetic annual peak discharges for a 10-year period (1960-69) for the 30 sites used in the regression analysis.
$\overline{I_V}$ = 0.252	The average index of variability ($\overline{I_V}$) was computed by averaging the standard deviations of the logarithms of the synthetic annual peak discharges of the 30 sites.
$\overline{C_S}$ = -0.360	The average coefficient of skew ($\overline{C_S}$) was computed by averaging the skew coefficients of the logarithms of the synthetic annual peak discharges of the 30 sites.

Length of record is also a factor in the computation of SEP. Because 80 years of synthetic data are not equal to 80 years of observed data, it was necessary to compute the equivalent length of record of the synthetic data for use in the computation of SEP. The following equivalent lengths of record for the corresponding recurrence intervals were computed based on methods and information presented by Lichty and Liscum (1978) and Hardison (1971).

Table 7.--Equations for estimating peak discharges of small urban streams in Ohio

Equation number	Equation	SER (in log units)	SER (in percent)	SEP (in percent)
(1)	$UQ_2 = 158(A)^{0.68}(SL)^{0.23}(EL)^{0.63}(13-BDF)^{-0.45}$	0.1557	±37	±36
(2)	$UQ_5 = 179(A)^{0.73}(SL)^{0.31}(EL)^{0.83}(13-BDF)^{-0.38}$	.1564	±37	±38
(3)	$UQ_{10} = 192(A)^{0.76}(SL)^{0.34}(EL)^{0.91}(13-BDF)^{-0.34}$	.1580	±38	±39
(4)	$UQ_{25} = 209(A)^{0.78}(SL)^{0.37}(EL)^{1.00}(13-BDF)^{-0.29}$	.1605	±38	±40
(5)	$UQ_{50} = 221(A)^{0.80}(SL)^{0.39}(EL)^{1.05}(13-BDF)^{-0.26}$	.1627	±39	±40
(6)	$UQ_{100} = 234(A)^{0.81}(SL)^{0.40}(EL)^{1.09}(13-BDF)^{-0.23}$	.1650	±39	±41

where:

UQ_x = urban peak discharge with recurrence interval of x years (cubic feet per second),

A = drainage area (square miles),

SL = main-channel slope (feet per mile),

EL = average basin elevation index (thousands of feet), and

BDF = basin development factor (scale of 0-12).

The values of dependent (UQ_x) and independent (A, BDF, SL, EL) variables used in the regression analysis are presented in table 6.

Recurrence interval (in years)	Equivalent length of record of synthetic data (in years)
2	5
5	9
10	14
25	19
50	21
100	21

The inclusion of A, SL, and BDF in equations 1 through 6, as well as the signs and magnitudes of their regression coefficients, appear to be appropriate from a hydrologic standpoint. However, the appropriateness of EL is not as obvious.

In an effort to explain the significance of EL, a multiple regression analysis was performed in which EL was related to several physical and climatic independent variables. An equation was produced with a surprisingly low standard error of 9 percent. Two independent variables, P and RI2-30, were significant at the 1-percent level. The other significant variables--SL, LT, and BDF--were significant at a level of less than 8 percent. Apparently, EL is a good indicator of the combined effects of rainfall patterns and several basin characteristics on peak discharges.

All independent variables in equations 1 through 6 (table 7) are statistically significant at the 1-percent level, with the following exceptions. Slope (SL) was significant at the 3-percent level in the UQ_2 equation. Average basin elevation index (EL) was significant at the 10-percent level in the UQ_2 equation, the 4-percent level in the UQ_5 equation, and the 2-percent level in the UQ_{10} and UQ_{25} equations. The significance level of basin development factor, BDF, decreased to the 2-percent level in the UQ_{50} equation, and to the 4-percent level in the UQ_{100} equation.

Sensitivity Analysis

The evaluation of BDF for a site may be somewhat subjective. The computation of A, SL, and EL from topographic maps is also subject to errors in measurement or judgment. A sensitivity analysis was performed to illustrate the effects of random errors in the independent variables on the computations. The means of the four independent variables were calculated to be:

A = 0.778 mi^2 SL = 92.8 ft/mi EL = 0.863 (1,000 ft) BDF = 7.97

These values were substituted into the six regression equations. Each independent variable was then varied by 5-percent increments from -50 percent to +50 percent of its mean, while the values of other variables were held constant. The percentage of change in the independent variable was then plotted against the percentage of change in the computed discharge. The results are presented in figure 6. (Because all six plots were quite similar, only the UQ_2, UQ_{25}, and UQ_{100} plots are shown.)

The sensitivity of BDF decreases for floods with higher recurrence intervals, as was evident in the evaluation of the significance levels of independent variables. This decreased significance of BDF also can be illustrated by plotting urban and rural flood-frequency curves together (fig. 5), in which case the curves tend to converge (or cross) at higher recurrence intervals. The tendency for BDF to have less effect at higher recurrence intervals can be explained. Impervious area (IA), which is closely related to BDF, tends to be less significant during large floods as soils become more impervious due to saturation. In addition, flood peaks of highly developed basins may show less of an increase during large floods because of temporary storage behind underdesigned culverts, bridges, and storm sewers.

It should also be noted that BDF becomes increasingly sensitive to positive errors in its value, which indicates that as a basin approaches full development (BDF = 8 to 12), any further development will tend to affect increases in peak discharges more than any previous development. Therefore, an accurate evaluation of BDF is more critical in the 8-to-12 range.

Tests for Intercorrelation and Bias

All significant variables were checked for intercorrelation. A high degree of intercorrelation between independent variables may affect the magnitude and sign of their regression coefficients, as well as erroneously reducing their statistical significance. Table 8 is a correlation matrix of the variables used in the peak discharge equations. All correlation coefficients between independent variables are between 0.50 and -0.50, except for A and SL (-0.68). Multicollinearity tests for A and SL indicated a minimal effect on the magnitudes and signs of their regression coefficients and no appreciable effect on the predictive ability of the equations.

All equations were tested for parametrical and geographical bias. Parametrical bias was tested by plotting the residuals (differences between the observed and estimated peak discharge for a specific recurrence interval) against each of the independent variables for all basins. Visual inspection of the plots indicated that the signs and magnitudes of the residuals varied randomly within the ranges of the independent variables.

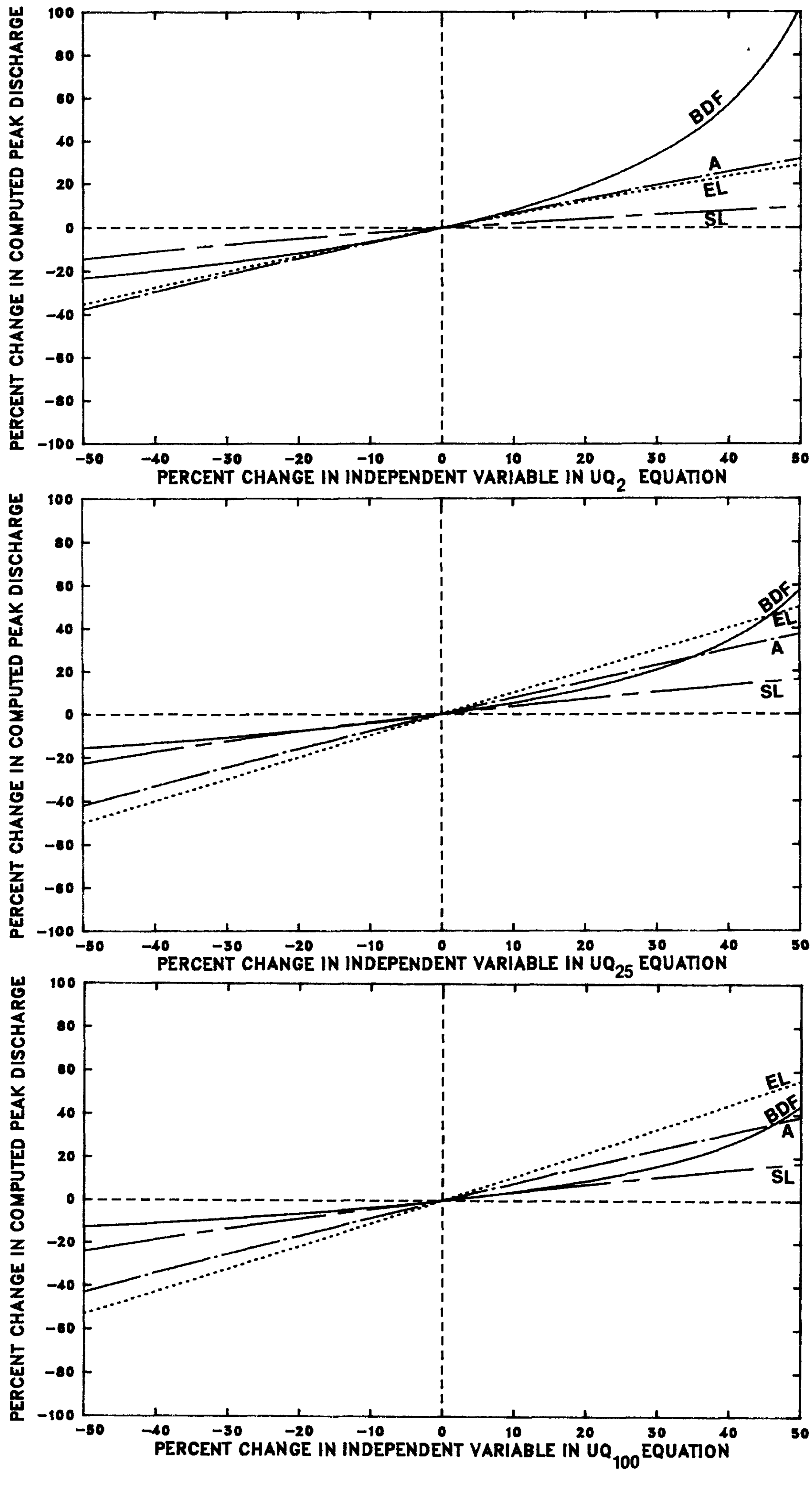

Figure 6.--Sensitivity analysis of the four independent variables A, SL, EL, and BDF.

Table 8.--Correlation matrix for significant variables

[All variables are in log units.]

Variable	A	13-BDF	SL	EL	UQ2	UQ100
A--------------	1.00	0.18	-0.68	-0.12	0.73	0.75
13-BDF---------	--	1.00	0.17	0.37	-0.34	-0.22
SL-------------	--	--	1.00	0.21	-0.31	-0.23
EL-------------	--	--	--	1.00	0.24	0.27
UQ2------------	--	--	--	--	1.00	0.94
UQ100----------	--	--	--	--	--	1.00

Residuals also were checked against the location of BDF within the basin, based on the assumption that basin development in the upper end of the basin increases peak discharges more than basin development in the lower end. Five sites were significantly more developed in either the upper or lower end of the basin. The residuals for these five sites indicated no apparent trend or bias.

Geographical bias was tested by plotting the residuals at the location of the corresponding site and checking for any tendencies to overestimate or underestimate in any areas of the state or within any cities. The residuals varied randomly both throughout the state and within each city. These tests indicate no apparent parametrical or geographical bias.

Comparison to Nationwide Urban Equations

The nationwide urban flood study related flood-frequency data to basin characteristics at 199 urban sites in 31 states using techniques similar to those used in this study. Table 9 compares the reported average standard errors of regression (SER) and average standard errors of prediction (SEP) of the 3-parameter and preferred 7-parameter nationwide equations (p. 19) to the SERs and SEPs of the equations developed in this study.

The 3-parameter and preferred 7-parameter nationwide equations were then used to estimate the 2-year, 10-year, and 100-year peak discharges of the 30 urban sites used in this study. RQ_x (an independent variable in both the 3-parameter and 7-parameter equations) was computed using equations developed by Webber and Bartlett (1977) for estimating rural peak discharges in Ohio. Table 10 presents the results of an error analysis. The mean error ($\bar{X}$), standard deviation of the errors (s), and root mean squared error (RMS) were computed for each equation (methods of computation are defined in the glossary).

The mean error ($\bar{X}$) is an indication of the bias present in the equations when applied to Ohio data. The results of the error analysis show that all the nationwide equations tested have a positive average error. Student's t-test indicates that these positive errors are statistically significant at the 5-percent level for the 3-parameter 10-year and 100-year equations, and at the 1-percent level for all the 7-parameter equations.

The root mean squared error (RMS) may be compared with the standard errors of prediction (SEP) reported for the nationwide equations. The RMS errors are about the same (± 1 percent) as the SEPs reported for the 2-year equations (both 3- and 7-parameter) and slightly higher (5 to 9 percent) for the 10-year and 100-year equations.

Table 9.--Comparison of average standard errors

Recurrence interval (in years)	Average standard errors of regression (in percent)			Average standard errors of prediction (in percent)		
	Ohio	Nationwide 3-parameter[1]	Nationwide 7-parameter[1]	Ohio	Nationwide 3-parameter	Nationwide 7-parameter
2	±37	±43	±38	±37	±44	±44
10	±38	±41	±38	±39	±43	±45
100	±39	±46	±44	±41	±49	±53

[1]Sauer and others, 1983

Table 10.--Error analysis of nationwide equations applied to Ohio data

[X, mean error; s, average standard deviation; RMS, root mean squared error]

Recurrence interval (in years)	Nationwide 3-parameter			Nationwide 7-parameter		
	X	s	RMS	X	s	RMS
	log units/percent			log units/percent		
2	0.0525/13	0.1729/±41	0.1807/±43	0.0804/20[b]	0.1666/±40	0.1850/±45
10	.0691/17[a]	.1839/±44	.1965/±48	.1036/27[b]	.1854/±45	.2124/±52
100	.0882/23[a]	.2059/±50	.2240/±55	.1322/36[b]	.2102/±51	.2483/±62

[a]Indicates that positive average errors are statistically significant based on Student's t-test at 5-percent level of significance.

[b]Indicates that positive average errors are statistically significant based on Student's t-test at 1-percent level of significance.

When compared with the SEPs of the Ohio equations, the RMS errors of the nationwide equations are higher (6 to 21 percent). This, combined with the overall positive bias of the nationwide equations, indicates that the Ohio equations are more precise than the nationwide equations for estimating peak discharges of small (less than 4 square miles) urban streams in Ohio.

Analysis of Flood Volume as a Function of Peak Discharge and Basin Characteristics

The approximate volume for a given peak discharge can be valuable information when designing hydraulic structures such as culverts and detention basins where storage may be part of the design criteria. A data set of the observed volumes of direct runoff and observed peak discharges was compiled from 153 of the largest (highest peak discharge) storms from the 30 study sites. All events with a maximum observed peak discharge value greater than the corresponding synthetic UQ_2 were selected. A maximum of eight and minimum of four events per site were included in the data set. If a station had less than four events greater than the corresponding synthetic UQ_2, the four largest (highest peak discharge) events were selected. Hydrograph shape was not a criterion in the selection process, thus, a wide variety of hydrograph shapes were included. A regression analysis using techniques similar to those in the peak-discharge analysis resulted in the following equation:

$$V = 147{,}000(Q_p)^{0.69}(A)^{0.26}(SL)^{-0.54}(13-BDF)^{0.30} \qquad (7)$$

where:

V = volume of direct runoff (ft^3)

Q_p = urban peak discharge (ft^3/s)

A = drainage area (mi^2)

SL = main-channel slope (ft/mi)

BDF = basin-development factor (scale of 0 to 12).

The average standard error of regression is ±52 percent, and all independent variables were significant at the 1-percent level. Bias tests indicated no apparent parametrical or geographical bias.

For a given basin, the shape (or width) of a runoff hydrograph is largely dependent upon the shape of the hyetograph (plot of rainfall intensity against time) from which it resulted. A high-intensity storm (thunderstorm) will tend to produce a steep, sharp-crested hydrograph. A low-intensity storm (frontal storm) will tend to produce a broad, flat-crested hydrograph. Thus, for a given basin, one might expect a large variation in hydrograph shapes and volumes for storms of the same peak discharge. Therefore, an average standard error of ± 52 percent seems appropriate. Equation 7 may be used to estimate the average (or typical) flood volume of direct runoff associated with a given peak discharge.

The inclusion of Q_p, A, SL, and BDF in equation 7, along with the signs and magnitudes of their regression coefficients, seems to be hydrologically valid. Increases in Q_p or A would result in an increase in V. Increases in SL or BDF would result in a shorter basin lagtime, thereby producing a steeper, more sharp-crested hydrograph. For a given peak discharge (Q_p), a steeper hydrograph would encompass a smaller flood volume. Hence, increases in SL or BDF would result in a decrease in V.

Equation 7 may be used to estimate the volume (V) for a given peak discharge (Q_p) at gaged or ungaged sites. The given peak discharge may be estimated or may be a known peak discharge from some other source. If the peak discharge is to be estimated, equations 1 through 6 (table 7, p. 22) may be applied. To illustrate the effects of errors in the estimated peak discharge on the computation of volume, a sensitivity analysis was performed. The means of the four independent variables from the 153 storms used in the regression analysis were calculated to be:

$Q_p = 207\ ft^3/s$ $\quad$ $A = 0.741\ mi^2$ $\quad$ $SL = 92.2\ ft/mi$ $\quad$ $BDF = 8.52$

These values were substituted into the volume equation. Q_p was then varied by 10-percent increments from -50 percent to +50 percent of its mean, while the values of A, SL, and BDF were held constant. The results are tabulated below.

Percentage change in Q_p	-50	-40	-30	-20	-10	0	+10	+20	+30	+40	+50
Percentage change in V	-38	-30	-22	-14	-7	0	+7	+13	+20	+26	+32

Analysis of Basin Lagtime as a Function of Basin Characteristics

As stated previously, lagtime, or basin response time, generally is defined as the time elapsed from the centroid of the rainfall excess to the centroid of the resultant runoff hydrograph. It is useful for estimating the shape of a runoff hydrograph for a given peak discharge. Estimated basin lagtime (LT) and peak discharge (Q_p) may be applied to a dimensionless hydrograph to generate a typical (average) flood hydrograph for the given peak discharge, as is demonstrated later in this report.

Lagtime for the 30 basins was computed as KSW + 1/2 TC, a relationship previously defined by Kraijenhoff van de Leur (1966), where KSW and TC (table 2) are those parameter values computed in the final model calibrations for each site. Previous investigators have related basin lagtime (LT) to main-channel length (L) and main-channel slope (SL) with the independent variable taking the form $L/\sqrt{SL}$. For this study, $L/\sqrt{SL}$ was tested in the lagtime regression analysis, as well as L/SL, L/SL^2, and the other independent variables listed on pages 13 and 14.

The following equation was derived by multiple-regression analysis:

$$LT = 1.07(L/\sqrt{SL})^{0.54}(13\text{-}BDF)^{0.42} \qquad (8)$$

where:

LT = lagtime (hours),

L = main-channel length (mi),

SL = main-channel slope (ft/mi), and

BDF = basin-development factor (scale of 0 to 12).

The average standard error of regression is ±48 percent, and both independent variables are statistically significant at the 1-percent level. Bias tests indicated no apparent parametrical or geographical bias.

In the nationwide urban flood study, Sauer and others (1983) developed a lagtime equation for nationwide use having the same independent variables as equation 8, only slightly different values for the intercept and regression coefficients, and a standard error of regression of ±76 percent.

ESTIMATING PEAK DISCHARGES AT UNGAGED URBAN SITES

Limitations of Method

The equations developed for estimating peak discharge are applicable to small urban basins in Ohio with basin characteristics similar to the basin characteristics of the sites used in the regression analysis. The following table indicates the ranges of the basin characteristics of the study sites.

Basin charac-teristic	Minimum	Maximum	Units
A	0.026	4.09	square miles
SL	8.00	462	feet per mile
EL	.622	1.21	thousands of feet
BDF	0	12	(scale of 0 to 12)

It is not recommended that the equations be applied to basins with characteristics outside of these ranges, as the standard errors of the results may be considerably higher.

All study sites were chosen to have little or no (less than 1 percent) basin storage (ST). The equations should not be applied to streams where floodflows are significantly affected by storage or where culverts, bridges, or storm sewers may significantly reduce peak discharges by temporarily storing water behind them.

For basins where the basin-development factor is low (less than 5) the peak discharge estimated from the urban equations may be close to or even less than the peak discharge estimated from the equations developed by Webber and Bartlett (1977) for use in rural areas. For BDFs of less than 5, the peak discharge could be estimated from both the urban and rural equations, and judgment exercised in evaluating the results.

A major assumption of this study is that annual peak discharges of small urban streams in Ohio are caused by rainfall, usually summer thunderstorms or large spring and fall frontal storms. Data were collected and analyzed accordingly. The equations, therefore, should not be applied to streams where annual peak discharges are likely to be caused by snowmelt.

Computation of Independent Variables

The values of the four independent variables are entered into the appropriate regression equation to compute the peak discharge for the desired recurrence interval. They may be determined as follows:

1. A--Drainage area, in square miles, as computed (by planimeter, digitizer, grid method, and so forth) from U.S. Geological Survey 7.5-minute topographic quadrangle maps (fig. 7). If it is suspected that the natural drainage area has been altered by urban development (such as storm-sewer lines crossing the natural basin divide), sewer maps may also be needed to accurately compute the drainage area (fig. 8).

2. SL--Main-channel slope, in feet per mile, is computed as the difference between the elevations (in feet) at 10 and 85 percent of the main-channel distance from the ungaged site to the basin divide, divided by the channel distance (in miles) between the two points, as determined from topographic maps and (or) sewer maps (fig. 7).

3. EL--Average basin-elevation index, in thousands of feet above NGVD of 1929, computed by averaging the elevations at the 10- and 85-percent distance points along the main channel, as determined from topographic maps and (or) sewer maps (fig. 7).

4. BDF--Basin-development factor indicates, on a scale from 0 to 12, the extent of urban development within the basin. The following description of how to determine BDF is based on information in a report by Sauer and others (1983). The drainage area is subdivided into thirds (lower, middle, and upper) by drawing two lines across the basin approximately perpendicular to the main channel and principal tributaries (fig. 9). Traveltimes for streams within each third should be about equal, which usually results in the lines resembling arcs whose common center is at the ungaged site. The subdivision generally can be drawn by eye, as precise measurement is not necessary; however, complex basin shapes and drainage patterns may require more scrutiny.

   Then, within each third, four aspects of the drainage system are evaluated and each assigned a code as follows (Sauer and others, 1983):

   1. Channel improvements.--If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a code of 1 is assigned. Any or all of

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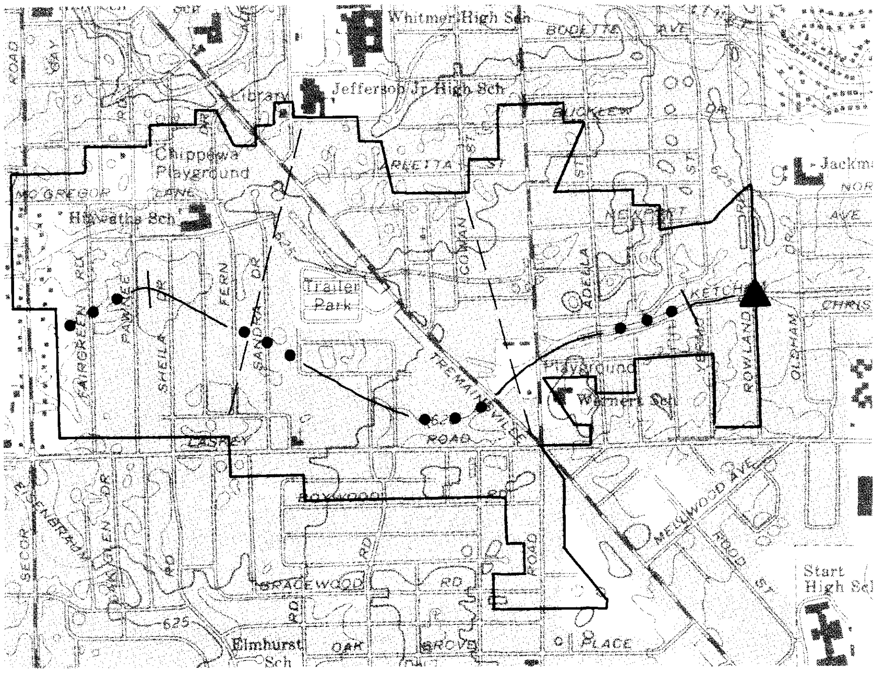
41°42'30"



Base from U.S. Geological Survey
Toledo 1:24,000, 1980

0 — 1 MILE
0 — 1 KILOMETER

EXPLANATION

▲ Study site

——— Drainage area boundary ($A=.839\ mi^2$).

— — — Lines subdividing the basin into thirds for determining the basin development factor (BDF=10).

• • •——┼ Main drainage channel with cross marks indicating the 10- and 85-percent distance points for computing the main-channel slope (SL=13.0 ft./mi.) and average basin elevation index (EL=.622 1000-ft.)

Figure 7.--Ketchum Ditch at Toledo, Ohio (site 04176870).

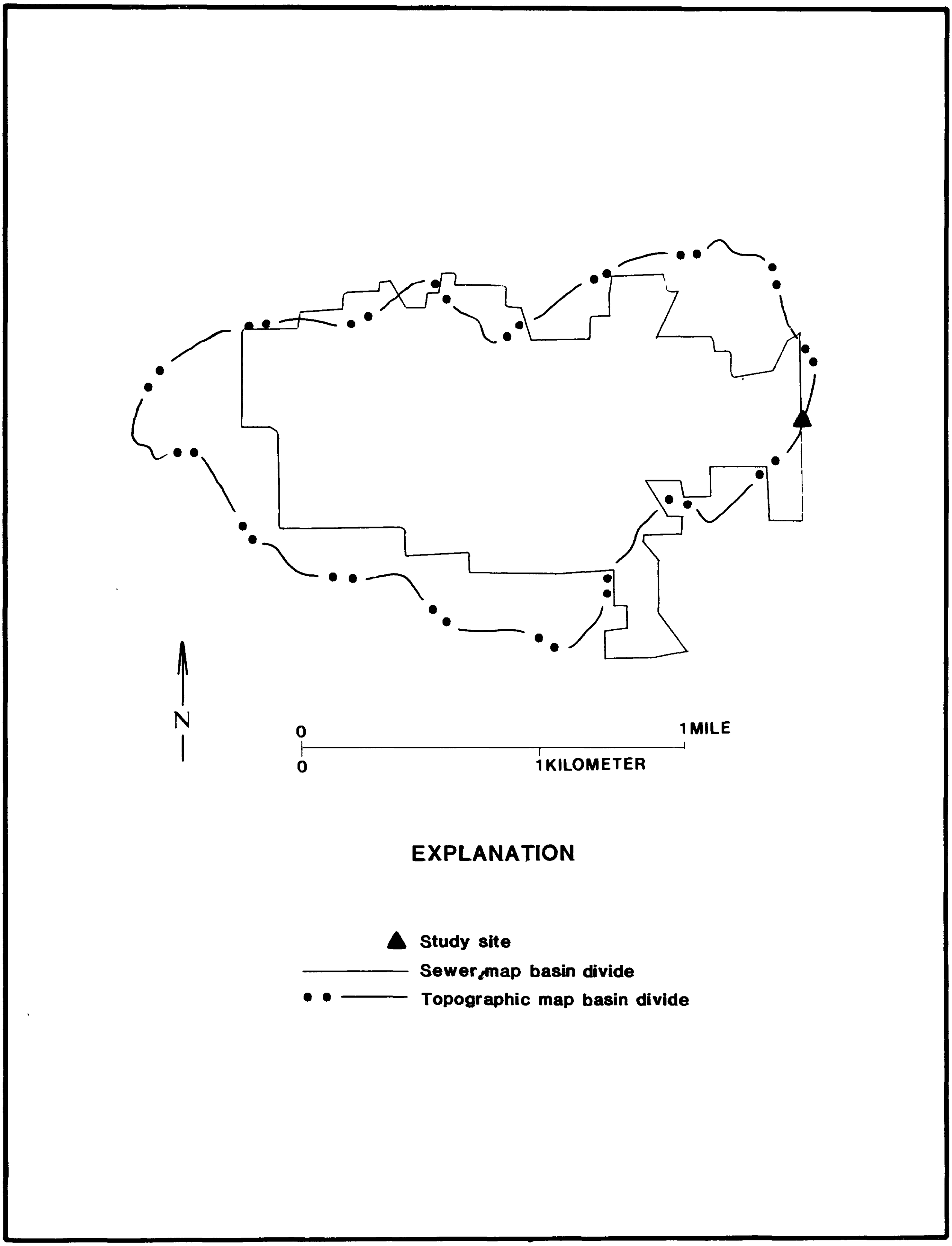


Figure 8.--Comparison of topographic-map and sewer-map basin divides at Ketchum Ditch, Toledo, Ohio.

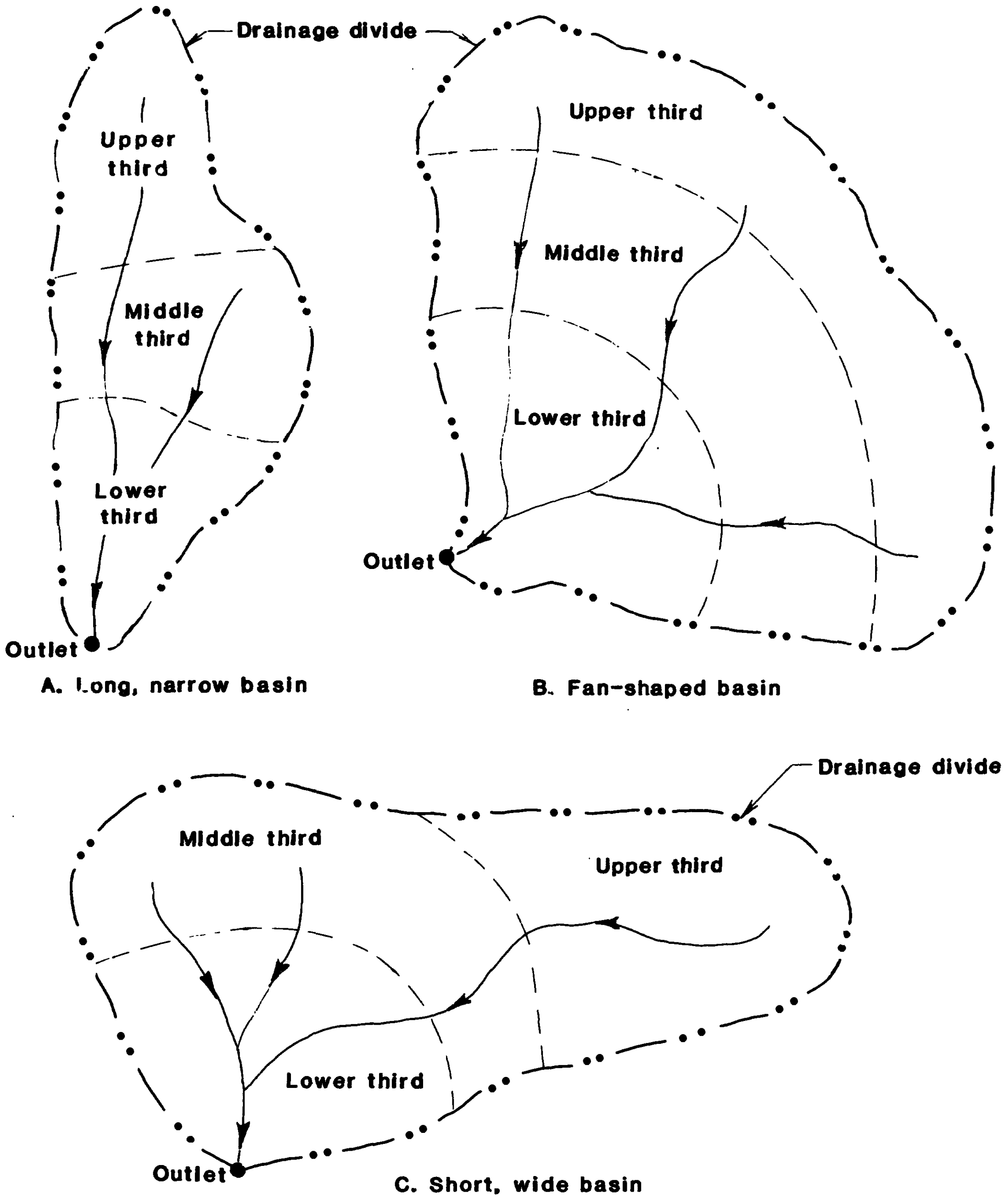


Figure 9.—Schematic diagrams of typical drainage-basin shapes and subdivision into thirds. (From Sauer and others, 1983).

these improvements would qualify for a code of 1. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of zero is assigned.

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

3. Storm drains or storm sewers.--Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Many of these drains empty into open channels; however, in some basins they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a subarea (third) consists of storm drains, then a code of 1 is assigned to this aspect; if less than 50 percent of the secondary tributaries consists of storm drains, then a code of zero is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, then the aspects of channel improvements and channel linings would also be assigned a code of 1.

4. Curb-and-gutter streets.--If more than 50 percent of a subarea (third) is urbanized (covered by residential, commercial, and (or) industrial development), and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, then a code of 1 would be assigned to this aspect. Otherwise, it would receive a code of zero. Drainage from curb-and-gutter streets frequently empties into storm drains.

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

Computation of Discharge

The following procedure should be used for estimating peak discharges of small urban streams in Ohio.

1. Determine the values of A, SL, EL, and BDF as described in "Computation of Independent Variables" (p. 33-9).

2. Check that the characteristics of the basin meet the criteria described in "Limitations of Method" (p. 32-3).

3. Select the appropriate equation from table 7 (p. 22) for the desired recurrence interval.

4. Substitute the values of A, SL, EL, and BDF into the equation.

5. Compute the peak discharge.

Example

Estimate the peak discharges for the 25-year and 100-year floods for Ketchum Ditch at Rowland Road in Toledo, Ohio (fig. 7).

1. The following basin characteristics are determined:

$A = 0.839 \text{ mi}^2$

$SL = 13.0 \text{ ft/mi}$

$EL = 0.622 \text{ (1,000 ft)}$

$BDF = 10$

Figure 8 illustrates the irregularity of the drainage area boundary and nonconformity with the natural basin divide. The location of the boundary was determined from sewer maps.

2. The basin characteristics meet the criteria described in "Limitations of Method."

3. The appropriate equations to be applied from table 7 (p. 22) are:

$$UQ_{25} = 209(A)^{0.78}(SL)^{0.37}(EL)^{1.00}(13\text{-}BDF)^{-0.29} \quad (4)$$

$$UQ_{100} = 234(A)^{0.81}(SL)^{0.40}(EL)^{1.09}(13\text{-}BDF)^{-0.23} \quad (6).$$

4. The basin characteristics are substituted into the equations:

$$UQ_{25} = 209(0.839)^{0.78}(13.0)^{0.37}(0.622)^{1.00}(13\text{-}10)^{-0.29}$$

$$UQ_{100} = 234(0.839)^{0.81}(13.0)^{0.40}(0.622)^{1.09}(13\text{-}10)^{-0.23}.$$

5. The estimated peak discharges are:

$$UQ_{25} = 213 \text{ ft}^3/\text{s}$$

$$UQ_{100} = 262 \text{ ft}^3/\text{s}.$$

ESTIMATING FLOOD VOLUMES AT UNGAGED URBAN SITES

Equation 7:

$$V = 147{,}000(Q_p)^{0.69}(A)^{0.26}(SL)^{-0.54}(13\text{-}BDF)^{0.30} \quad (7)$$

may be used for estimating the volumes of floods having peak discharges between 20 and 1,060 ft^3/s (the range of peak discharges used in the volume regression analysis). The limitations of the method and the computation of the independent variables are the same as those for estimating peak discharges.

Computation of Volume

The following procedure should be used for estimating the flood volumes associated with peak discharges (Q_p) of small urban streams in Ohio.

1. Determine the values of A, SL, and BDF as described in "Computation of Independent Variables" (p. 33-9).

2. Check that the characteristics of the basin meet the criteria described in "Limitations of Method" (p. 32-3), and that Q_p is between 20 and 1,060 ft^3/s.

3. Substitute the values of Q_p, A, SL, and BDF into equation 7.

4. Compute the volume.

Example

Estimate the volume associated with the peak discharge (Q_p) of the estimated 100-year flood for Ketchum Ditch at Rowland Road in Toledo, Ohio (fig. 7), where:

$$UQ_{100} = 262 \text{ ft}^3/\text{s}$$

1. The following basin characteristics are determined:

$$A = 0.839 \text{ mi}^2,$$

$$SL = 13.0 \text{ ft/mi, and}$$

$$BDF = 10$$

2. The basin characteristics meet the criteria described in "Limitations of Method" (p. 32-3), and Q_p is between 20 and 1,060 ft^3/s.

3. The basin characteristics and Q_p are substituted into equation 7:

$$V = 147{,}000(Q_p)^{0.69}(A)^{0.26}(SL)^{-0.54}(13-BDF)^{0.30}$$

$$V = 147{,}000(262)^{0.69}(0.839)^{0.26}(13.0)^{-0.54}(13-10)^{0.30}.$$

4. The estimated volume associated with a peak discharge of 260 ft^3/s is:

$$V = 2{,}280{,}000 \text{ ft}^3.$$

ESTIMATING HYDROGRAPH SHAPES AT UNGAGED URBAN SITES

The shape of a runoff hydrograph can be a useful tool for designing detention basins or for using embankment ponding to reduce culvert size. A technique is described here for estimating the hydrograph shape for a specified peak discharge. Estimated basin lagtime (LT) and peak discharge (Qp) are applied to a dimensionless hydrograph to generate a typical (average) flood hydrograph for the given peak discharge. If the peak discharge is to be estimated, equations 1 through 6 (table 7, p. 22) may be applied.

Dimensionless Hydrograph

A dimensionless hydrograph is essentially a representative hydrograph shape for which the discharge is expressed as the ratio of discharge to peak discharge and the time as the ratio of time to lagtime (fig. 10). It is developed by averaging typical hydrographs from a variety of basins. The hydrographs used in the analysis generally are single-peak events of relatively short duration. Previous investigators have developed several dimensionless hydrographs, all of which are very similar.

The dimensionless hydrograph used here was developed by Stricker and Sauer (1982) in a nationwide urban hydrograph study. It was derived on the assumption that the duration of the rainfall excess was approximately equal to one-third of the basin lagtime. The coordinates are listed in table 11 and plotted in figure 10. The Stricker-Sauer dimensionless hydrograph was verified for use in Ohio by applying it to 10 of the 30 sites used in this study. The 10 sites were selected to be equally distributed throughout the State and to represent the full range of values of drainage area and basin lagtime. Simulated hydrographs were compared to observed hydrographs at each of the 10 sites. The simulated hydrographs were generated by applying the average station lagtime and peak discharge of the observed hydrograph to the Stricker-Sauer dimensionless hydrograph. The simulated and observed hydrographs compared well at all 10 sites with no tendency to overestimate or underestimate the width of the hydrographs. An example of this comparison is shown in figure 11.

Estimating Basin Lagtime

Equation 8:

$$LT = 1.07(L/\sqrt{SL})^{0.54}(13-BDF)^{0.42} \tag{8}$$

may be used for estimating the lagtime of small urban basins in Ohio. The limitations of the method are the same as those for estimating peak discharges.

The following procedure should be used for estimating basin lagtime.

1. Determine the value of L where L (main-channel length, in miles) is computed as the distance measured along the main channel from the ungaged site to the basin divide, as determined from topographic maps.

2. Determine the values of SL and BDF as described in "Computation of Independent Variables" (p. 33-9).

3. Check that the characteristics of the basin meet the criteria described in "Limitations of Method" (p. 32-3).

4. Substitute the values of SL and BDF into equation 8.

5. Compute the lagtime.

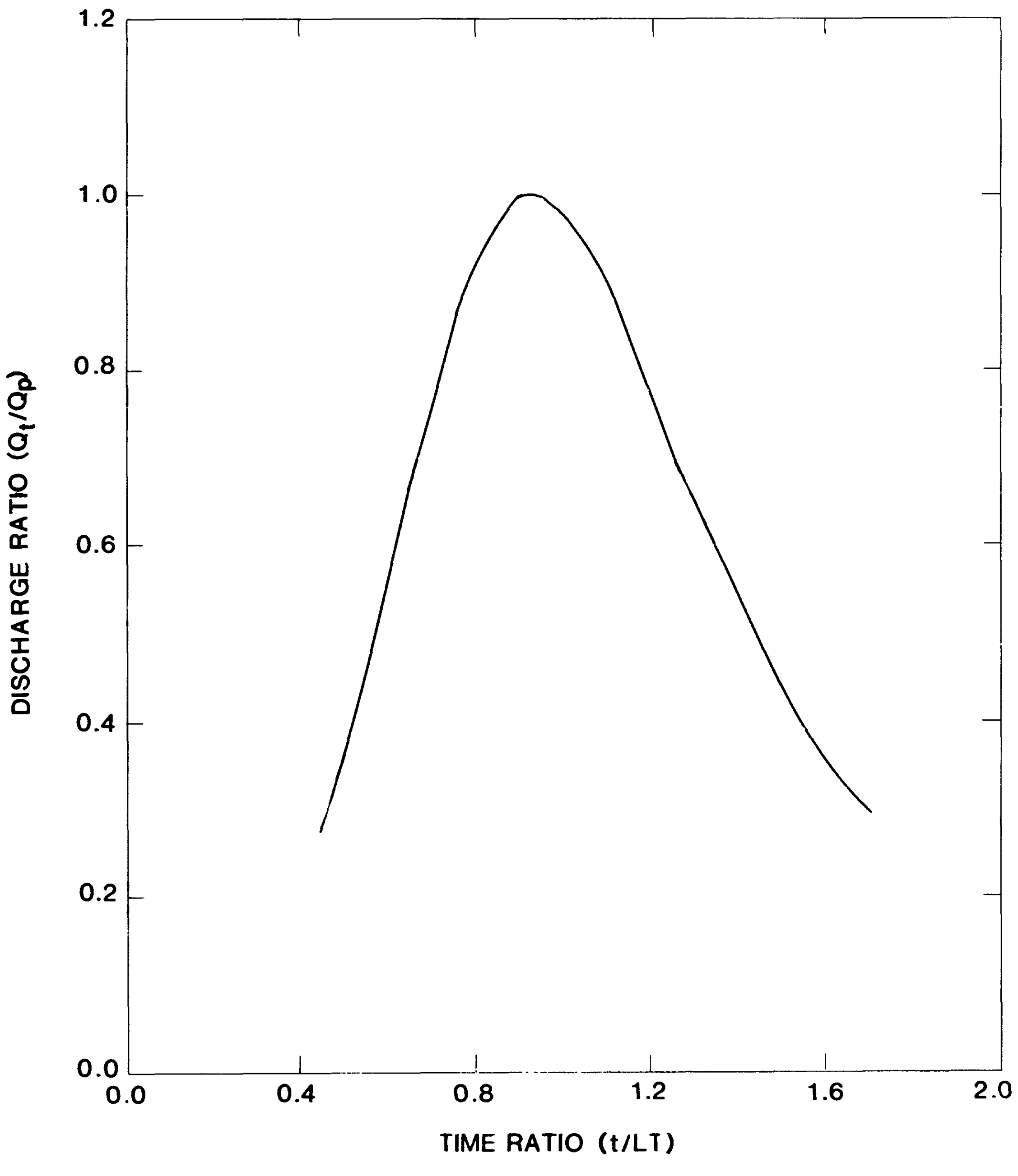


Figure 10.--Dimensionless hydrograph (Stricker and Sauer, 1982).

Table 11.--Time and discharge ratios of the dimensionless hydrograph

[Stricker and Sauer, 1982; t = time, in hours; LT = lagtime, in hours; Q_t = discharge, in cubic feet per second, at time t; and Q_p = peak discharge, in cubic feet per second]

Time ratio (t/LT)	Discharge ratio (Q_t/Q_p)
0.45	0.27
.50	.37
.55	.46
.60	.56
.65	.67
.70	.76
.75	.86
.80	.92
.85	.97
.90	1.00
.95	1.00
1.00	.98
1.05	.95
1.10	.90
1.15	.84
1.20	.78
1.25	.71
1.30	.65
1.35	.59
1.40	.54
1.45	.48
1.50	.44
1.55	.39
1.60	.36
1.65	.32
1.70	.30

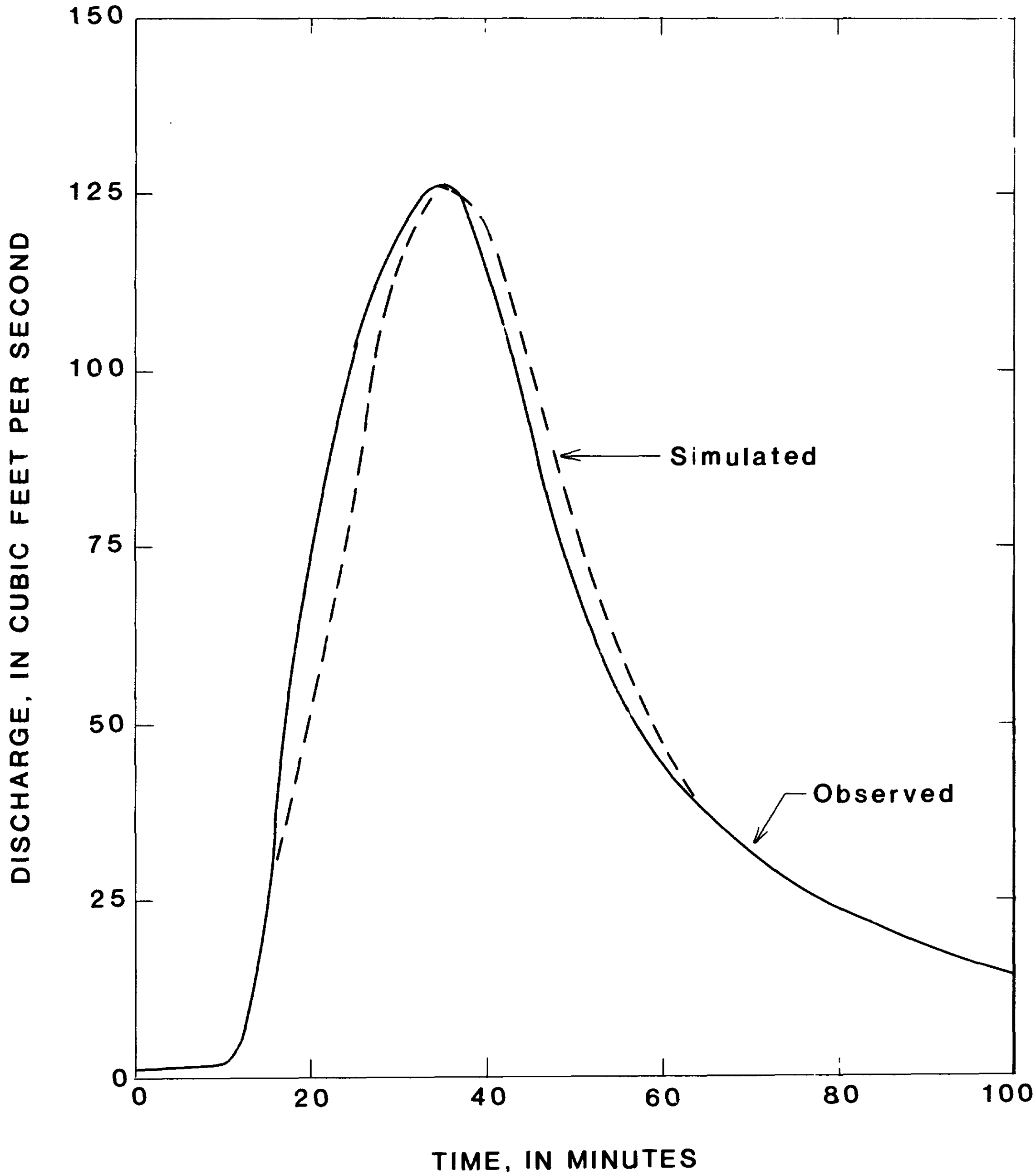


Figure 11 --Simulated and observed hydrographs for flood event of 5-14-83 for Charles Ditch at Boardman, Ohio.

Example

Estimate the basin lagtime for Ketchum Ditch at Rowland Road in Toledo, Ohio (fig. 7).

1. and 2. The following basin characteristics are determined:

$L = 1.54$ mi,

$SL = 13.0$ ft/mi, and

$BDF = 10$

3. The basin characteristics meet the criteria described in "Limitations of Method."

4. The basin characteristics are substituted into equation 8:

$$LT = 1.07(L/\sqrt{SL})^{0.54}(13\text{-}BDF)^{0.42}$$

$$LT = 1.07(1.54/\sqrt{13.0})^{0.54}(13-10)^{0.42}$$

5. The estimated basin lagtime is:

$LT = 1.07$ hours

Estimating Hydrograph Shape

The following procedure should be used for generating a representative hydrograph for a specific peak discharge.

1. If the peak discharge (Q_p) is to be estimated, use the procedure described in "Estimating Peak Discharges at Ungaged Urban Sites" (p. 32-40).

2. Estimate the basin lagtime (LT) using the procedure described in "Estimating Basin Lagtime" (p. 42).

3. Multiply each value of t/LT in table 11 by LT. These computed values are the time scale for the hydrograph (t = t/LT x LT).

4. Multiply each value of Q_t/Q_p in table 11 by Q_p. These computed values are the corresponding discharge values at time $t (Q_t = Q_t/Q_p \times Q_p)$.

5. Plot time (t) against discharge (Q_t).

Because the dimensionless hydrograph was developed from events of relatively short duration, the procedure outlined above will generate a hydrograph of relatively short duration. Hydrographs with the same peak discharge but with considerably longer duration (and hence, greater volume) also are likely to occur. Therefore, this procedure should only be used to generate a short duration (low-volume) hydrograph.

Example

Estimate the shape of the hydrograph of the estimated 100-year flood for Ketchum Ditch at Rowland Road in Toledo, Ohio (fig. 7), where:

$A = 0.839$ mi^2,

$SL = 13.0$ ft/mi,

$EL = 0.622$ (1,000 ft),

$BDF = 10$, and

$L = 1.54$ mi

1. The 100-year flood peak discharge is estimated using equation 6 (table 7, page 22).

$$UQ_{100} = 234(A)^{0.81}(SL)^{0.40}(EL)^{1.09}\ (13\text{-}BDF)^{-0.23} \qquad (6)$$

$$UQ_{100} = 234(0.839)^{0.81}(13.0)^{0.40}(0.622)^{1.09}(13\text{-}10)^{-0.23}$$

$$Q_p = UQ_{100} = 262 \text{ ft}^3/\text{s}.$$

2. The basin lagtime is estimated using equation 8.

$$LT = 1.07(L/\sqrt{SL})^{0.54}(13\text{-}BDF)^{0.42}$$

$$LT = 1.07(1.54/\sqrt{13.0})^{0.54}(13\text{-}10)^{0.42}$$

$$LT = 1.07 \text{ hours.}$$

3. Each value of t/LT in table 11 is multiplied by 1.07 hours. (Results are presented in table 12.)

4. Each value of Q_t/Q_p in table 11 is multiplied by 262 ft^3/s. (Results are presented in table 12.)

5. Time (t) versus discharge (Q_t) is plotted (fig. 12). The rising and falling limbs of the hydrograph may be extrapolated as indicated by the dashed lines.

Suppose Ketchum Ditch is known to flow over Rowland Road at 120 ft^3/s. It can be estimated from figure 11 that there would be flow over the road for approximately 1 hour during a 100-year flood.

Table 12.--Computation of coordinates of estimated hydrograph shape of 100-year flood for Ketchum Ditch at Rowland Road, Toledo, Ohio

[Refers to example on p. 47. ft^3/s, cubic feet per second.]

t/LT (from table 11)	x	LT (from step 2)	=	t time, in hours	Q_t/Q_p (from table 11)	x	Q_p (from step 1)	=	Q_t discharge, in ft^3/s
0.45		1.07		0.48	0.27		262		71
.50		1.07		.54	.37		262		97
.55		1.07		.59	.46		262		121
.60		1.07		.64	.56		262		147
.65		1.07		.70	.67		262		176
.70		1.07		.75	.76		262		199
.75		1.07		.80	.86		262		225
.80		1.07		.86	.92		262		241
.85		1.07		.91	.97		262		254
.90		1.07		.96	1.00		262		262
.95		1.07		1.02	1.00		262		262
1.00		1.07		1.07	.98		262		257
1.05		1.07		1.12	.95		262		249
1.10		1.07		1.18	.90		262		236
1.15		1.07		1.23	.84		262		220
1.20		1.07		1.28	.78		262		204
1.25		1.07		1.34	.71		262		186
1.30		1.07		1.39	.65		262		170
1.35		1.07		1.44	.59		262		155
1.40		1.07		1.50	.54		262		141
1.45		1.07		1.55	.48		262		126
1.50		1.07		1.60	.44		262		115
1.55		1.07		1.66	.39		262		102
1.60		1.07		1.71	.36		262		94
1.65		1.07		1.77	.32		262		84
1.70		1.07		1.82	.30		262		79

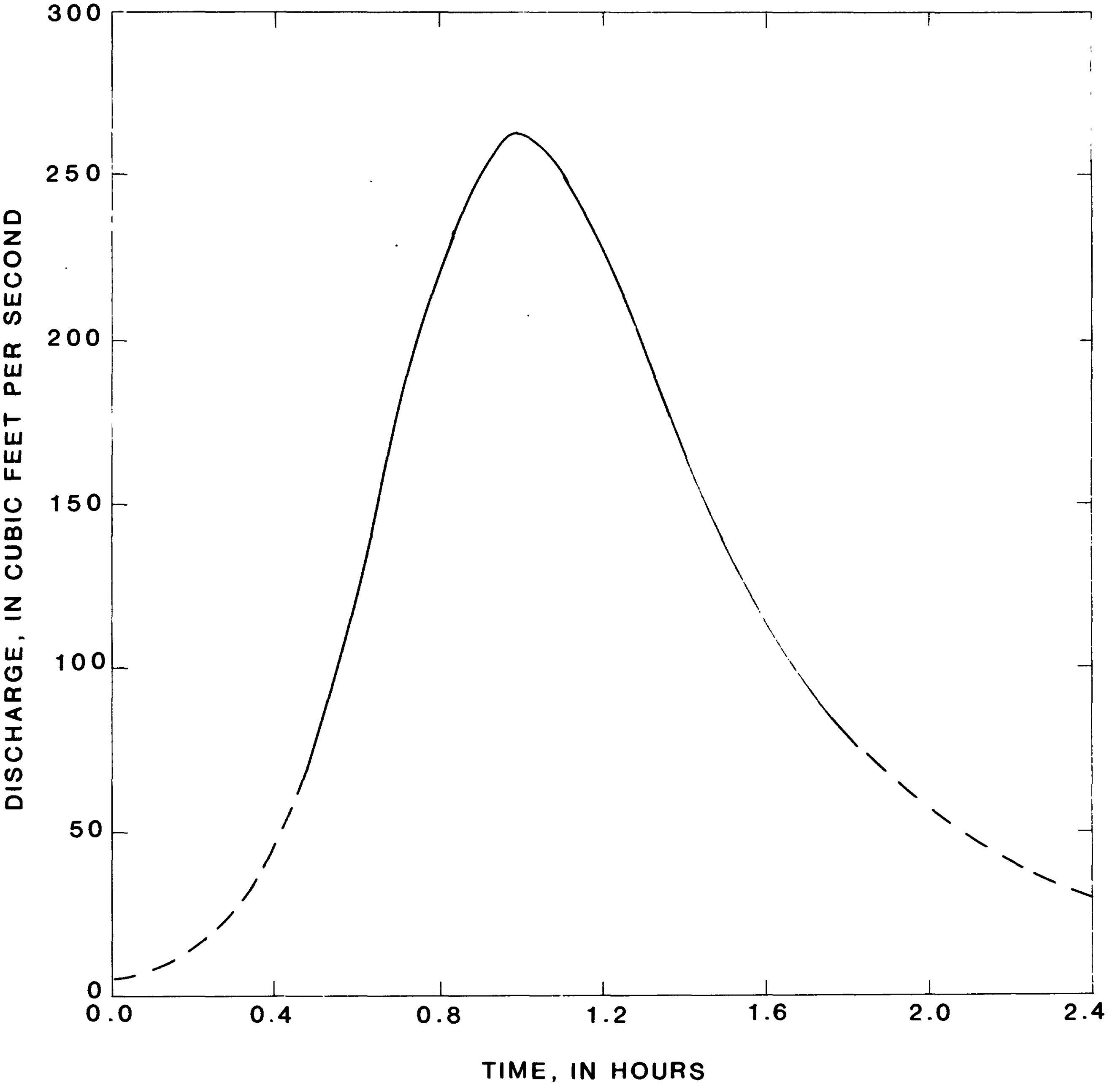


Figure 12.--Estimated hydrograph shape of 100-year flood for Ketchum Ditch at Rowland Road in Toledo, Ohio.

As an alternative to the volume equation (7) developed in this report, the flood volume associated with a peak discharge may be estimated by integrating under the estimated hydrograph. The volume equation will tend to estimate higher volumes, because it was developed from events of both long and short duration, whereas the dimensionless hydrograph was developed from events of relatively short duration only.

SUMMARY

An 11-year urban runoff study was conducted to evaluate the effects of urbanization and develop methods of estimating peak discharges, flood volumes, and hydrograph shapes of small urban streams in Ohio.

Five-minute rainfall-runoff data were collected for a period of 5 to 8 years at 30 small (less than 5 square miles) partly to fully urbanized basins distributed throughout the metropolitan areas of Ohio. A U.S. Geological Survey rainfall-runoff model was calibrated for each site. Long-term rainfall and evaporation records were used in conjunction with the calibrated models to synthesize a long-term (about 80 years) annual peak discharge record for each site. Each annual peak flow record was then fitted to a Log-Pearson Type III frequency curve.

Multiple-regression analysis was used to relate the flood-frequency data to various physical and climatic variables. Regression equations were developed for estimating peak discharges having recurrence intervals of 2, 5, 10, 25, 50, and 100 years. The significant independent variables were drainage area, main-channel slope, average basin-elevation index, and basin-development factor. Standard errors of regression and prediction range from ±37 percent to ±41 percent.

An equation was also developed to estimate the flood volume of a given peak discharge, where peak discharge, drainage area, main-channel slope, and basin-development factor were found to be the significant independent variables. The standard error of regression for the volume equation is ±52 percent.

An equation for estimating the lagtime of a basin was developed in which main-channel length divided by the square root of the main-channel slope ($L/\sqrt{SL}$) and basin-development factor were significant variables and the standard error of regression is ±48 percent. A technique was described for estimating the shape of a runoff hydrograph by applying a specific peak discharge and the estimated lagtime to a dimensionless hydrograph.

Examples of how to use the various regression equations and estimating techniques are presented.

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