Estimation of Peak-Frequency Relations, Flood Hydrographs, and Volume-Duration-Frequency Relations of Ungaged Small Urban Streams in Ohio

United States Geological Survey Water-Supply Paper 2432

Prepared in cooperation with the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration

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Estimation of Peak-Frequency Relations, Flood Hydrographs, and Volume-Duration-Frequency Relations of Ungaged Small Urban Streams in Ohio

By JAMES M. SHERWOOD

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Prepared ın cooperatıon with the Ohıo Department of
Transportatıon and the U S Department of Transportatıon,
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CONVERSION FACTORS

Multiply	By	To obtain
inch (in.)	25.4	millimeter
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
foot per mile (ft/mi)	0.1894	meter per kilometer
square inch (in^2)	6.452	square centimeter
square mile (mi^2)	2.590	square kilometer
cubic foot (ft^3)	0.02832	cubic meter
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

SYMBOLS

The following are definitions of selected symbols as they are used in this report; they are not necessarily the only valid definitions for these symbols.

A Drainage area (in square miles)—The drainage area that contributes surface runoff to a specified location on a stream, measured in a horizontal plane. Computed (by planimeter, digitizer, or grid method) from U.S. Geological Survey 7.5-minute topographic quadrangle maps. Sewer maps may be necessary to delineate drainage area in urban areas because sewer lines sometimes cross topographic divides.

BDF Basin-development factor—A measure of basin development that takes into account 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 text for a more complete description and method of computation.

d Duration of a maximum flood-volume or maximum rainfall event (in hours).

D Duration of a simulated flood hydrograph (in hours).

dRF_T *d*-hour *T*-year rainfall (in inches)—Maximum rainfall having a *d*-hour duration and *T*-year recurrence interval. Determined from U.S. Weather Bureau Technical Paper 40 (U.S. Department of Commerce, 1961).

dV_T *d*-hour *T*-year flood volume (in millions of cubic feet)—Maximum flood volume having a *d*-hour duration and *T*-year recurrence interval. Computed from frequency analysis of synthetic annual peak-volume data, or estimated from multiple-regression equations presented in this report.

EL Average basin elevation index (in thousands of feet above sea level)—Determined by averaging main-channel elevations at points 10 and 85 percent of the distance from a specified location on the main channel to the topographic divide, as determined from U.S. Geological Survey 7.5-minute topographic quadrangle maps.

IA Impervious area (in percent)—That part of the drainage area covered by impervious surfaces such as streets, parking lots, and buildings.

L Main-channel length (in miles)—Distance measured along the main channel from a specified location to the topographic divide, as determined from U.S. Geological Survey 7.5-minute topographic quadrangle maps.

LT Basin lagtime (in hours)—Time elapsed from the centroid of the rainfall excess (rainfall contributing to direct runoff) to the centroid of the resultant runoff hydrograph. *LT* for urban basins may be estimated from a multiple-regression equation presented in this report.

P Average annual precipitation (in inches)—Determined from Ohio Department of Natural Resources Water Inventory Report No. 28 (Harstine, 1991).

Q Discharge (in cubic feet per second).

Q_p Peak discharge (in cubic feet per second)—The maximum discharge of an observed or simulated flood hydrograph

Q_T Peak discharge (in cubic feet per second)—Peak discharge with recurrence interval of T years

RQ_T Rural peak discharge (in cubic feet per second)—The estimated rural peak discharge with recurrence of T years, as computed from regionalized regression equations developed by Koltun and Roberts (1990)

SEP Average standard error of prediction (in percent)—An approximation of the error associated with estimating a streamflow characteristic of a site not used in the regression analysis

SER Average standard error of regression (in percent)—A measure of the error associated with estimating a streamflow characteristic of a site used in the regression analysis

SL Main-channel slope (in feet per mile)—Computed as the difference in elevations (in feet) at points 10 and 85 percent of the distance along the main channel from a specified location on the channel to the topographic divide, divided by the channel distance (in miles) between the two points, as determined from U S Geological Survey 7 5-minute topographic quadrangle maps

ST Storage area (in percent)—That part of the contributing drainage area occupied by lakes, ponds, and swamps, as shown on U S Geological Survey 7 5-minute topographic quadrangle maps Temporary storage as a result of detention basins or ponding upstream of roadway embankments is not included

t Time (in hours)

T Recurrence interval (in years)—The average interval of time within which a given hydrologic event will be equaled or exceeded once

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

VQ_p Volume of hydrograph having peak discharge Q_p (in cubic feet)—The total flood volume computed by numerically integrating the total area under a simulated hydrograph with peak discharge Q_p VQ_p may also be directly computed for the Georgia dimensionless hydrograph (Inman, 1987) using an equation presented in this report

$VQ_T(t)$ Cumulative volume at time t (in cubic feet)—Computed by numerically integrating the area of a simulated hydrograph from time zero to time t

VQ_T Hydrograph volume of Q_T (in cubic feet)—The total flood volume computed by integrating the area under a simulated hydrograph having a peak discharge with a T-year recurrence interval (Q_T)

Estimation of Peak-Frequency Relations, Flood Hydrographs, and Volume-Duration-Frequency Relations of Ungaged Small Urban Streams in Ohio

By James M. Sherwood

Abstract

Methods are presented to estimate peak-frequency relations, flood hydrographs, and volume-duration-frequency relations of urban streams in Ohio with drainage areas less than 6.5 square miles. The methods were developed to assist planners in the design of hydraulic structures for which hydrograph routing is required or where the temporary storage of water is an important element of the design criteria. Examples of how to use the methods also are presented

The data base for the analyses consisted of 5-minute rainfall-runoff data collected for periods from 5 to 8 years at 62 small drainage basins located throughout Ohio. The U.S. Geological Survey rainfall-runoff model A634 was used and was calibrated for each site The calibrated models were used in conjunction with long-term (66–87 years) rainfall and evaporation records to synthesize a long-term series of flood-hydrograph records at each site. A method was developed and used to increase the variance of the synthetic flood characteristics in order to make them more representative of observed flood characteristics.

Multiple-regression equations were developed to estimate peak discharges having recurrence intervals of 2, 5, 10, 25, 50, and 100 years. The explanatory variables in the peak-discharge equations are drainage area, average annual precipitation, and basin-development factor. Average standard errors of prediction for the peak-frequency equations range from ±34 to ±40 percent

A method is presented to estimate flood hydrographs by applying a specific peak discharge and basin lagtime to a dimensionless hydrograph An equation was developed to estimate basin lagtime in which main-channel length divided by the square root of the main-channel slope ($L/\sqrt{SL}$) and basin-development factor are the explanatory variables and the average standard error of prediction is ±53 percent. A dimensionless hydrograph originally developed by the U.S. Geological Survey for use in Georgia was verified for use in urban areas of Ohio.

Multiple-regression equations were developed to estimate maximum flood volumes of *d*-hour duration and *T*-year recurrence interval (dV_T). Annual maximum flood-volume data for all combinations of six durations (1, 2, 4, 8, 16, and 32 hours) and six recurrence intervals (2, 5, 10, 25, 50, and 100 years) were analyzed The explanatory variables in the resulting 36 volume-duration-frequency equations are drainage area, average annual precipitation, and basin-development factor. Average standard errors of prediction for the 36 dV_T equations range from ±28 percent to ±44 percent.

Step-by-step examples show how to estimate (1) peak discharges for selected recurrence intervals, (2) flood hydrographs and compute their volumes, and (3) volume-duration-frequency relations of small, ungaged urban streams in Ohio. Volumes estimated by use of the volume-duration-frequency equations were compared with volumes estimated by integrating under an estimated hydrograph. Both methods yield similar

results for volume estimates of short duration, which are applicable to convective-type storm runoff. The volume-duration-frequency equations can be used to compute volume estimates of long and short duration because the equations are based on maximum-annual-volume data of long and short duration. The dimensionless-hydrograph method is based on flood hydrographs of average duration and cannot be used to compute volume estimates of long duration Volume estimates of long duration may be considerably greater than volume estimates of short duration and are applicable to runoff from frontal-type storms.

INTRODUCTION

Accurate estimates of flood characteristics are required for the efficient and safe design of riverine structures such as bridges and culverts Estimates of flood-peak discharges may be the main consideration for designs where flood flows are required to pass through the structure with minimal detention storage upstream from the structure If detention storage is a primary consideration, the designer also may require accurate estimates of the shape of the flood hydrograph and the magnitude of flood volumes having specific recurrence intervals Stringent stormwater-management regulations (Ohio Department of Natural Resources, 1981) and rising construction costs have increased the importance of detention storage as a design consideration For example, stormwater management guidelines may require a reduction in peak discharge to lessen the effects of flooding downstream. In addition, construction of a smaller diameter culvert could significantly reduce costs at sites where sufficient detention storage can be provided to allow adequate storage of water during large-volume floods

The estimated peak discharge and corresponding estimated flood hydrograph may be all the information needed for design situations in which some storage is required but is not considered to be a critical factor Estimated flood hydrographs also provide a means of routing floods with specific design peak discharges through a hydraulic structure, so that outflow peak discharges may be estimated In situations where the design peak outflow is required or desired to be considerably less than the design peak inflow, a significant volume of water must be temporarily stored upstream of the structure In this case, an estimate of volume for a design duration is needed

The volume computed by integrating the design discharge hydrograph is frequently used as an estimate of volume The dimensionless hydrographs used to estimate design hydrographs are usually developed from observed flood hydrographs having relatively high peak discharges and approximately average durations Consequently, when a flood hydrograph is estimated by use of a dimensionless hydrograph, the result is a sharp-crested, approximately average-duration hydrograph with a smaller volume than that for a hydrograph having the same peak discharge but longer duration Development of a longer-duration dimensionless hydrograph, which would contain more volume, is not feasible because of the high degree of variability in the shapes of long-duration hydrographs Furthermore, the actual shape of the hydrograph becomes less important in the design of a detention basin having a relatively small outlet and large storage capacity It is more important to estimate the relation between inflow volume and time. This relation, in combination with an estimate of the relation between outflow volume and time, can be used to estimate the relation between the required storage volume and time

The objective of this study is to develop multiple-regression equations for estimating relations between volume, duration, and frequency at ungaged small urban streams in Ohio This objective is accomplished by applying techniques developed and data collected as part of a concurrent rural volume-duration-frequency study (Sherwood, 1993) The data base for the analyses includes rainfall-runoff data collected at 30 urban sites from a previous study (Sherwood, 1986) and 32 rural sites from the concurrent rural volume-duration-frequency study

In the early stages of this study, a method was developed that should improve the accuracy of synthetic flood-frequency data It was subsequently decided to revise the previously published urban peak-frequency data (Sherwood, 1986) on the basis of this new method and develop and publish revised urban peak-frequency equations as part of this study All three studies were conducted by the U S Geological Survey (USGS) in cooperation with the Ohio Department of Transportation and the U.S Department of Transportation, Federal Highway Administration

Purpose and Scope

This report summarizes the methods of data collection and analysis used in this study, presents revised equations for estimating peak-frequency relations, and presents new equations for estimating volume-duration-frequency relations for small, ungaged urban streams in Ohio A method of estimating flood hydrographs by applying estimated basin lagtime and peak discharge to a dimensionless hydrograph also is presented This report supersedes the previous urban runoff report (Sherwood, 1986)

Examples of how to use the equations and how to estimate flood hydrographs also are presented. The equations and methods developed for this study are based on 5-minute rainfall-runoff data collected for a period from 5 to 8 years at 62 small (less than 6 5 square miles) basins distributed throughout Ohio The equations and methods presented are applicable to small urban streams in Ohio whose basin characteristics are similar to the basin characteristics of the 62 study sites

Previous Work and Approach to This Study

The work of previous investigators was used to evaluate and select the most appropriate approach to use in developing methods of estimating the following three flood characteristics addressed in this study

1. Peak discharge having a specific recurrence interval, for example, a 100-year flood theoretically would occur an average of once every 100 years, or have a 1-percent chance of occurrence in any given year
2. Flood hydrograph having a specific peak discharge, for example, the 50-year flood hydrograph is a plot of discharge against time, in which the peak discharge has a 50-year recurrence interval
3. Flood volume having a specific duration and recurrence interval, for example, a 4-hour, 100-year volume is the maximum flood volume during a 4-hour period that, theoretically, would occur an average of once every 100 years

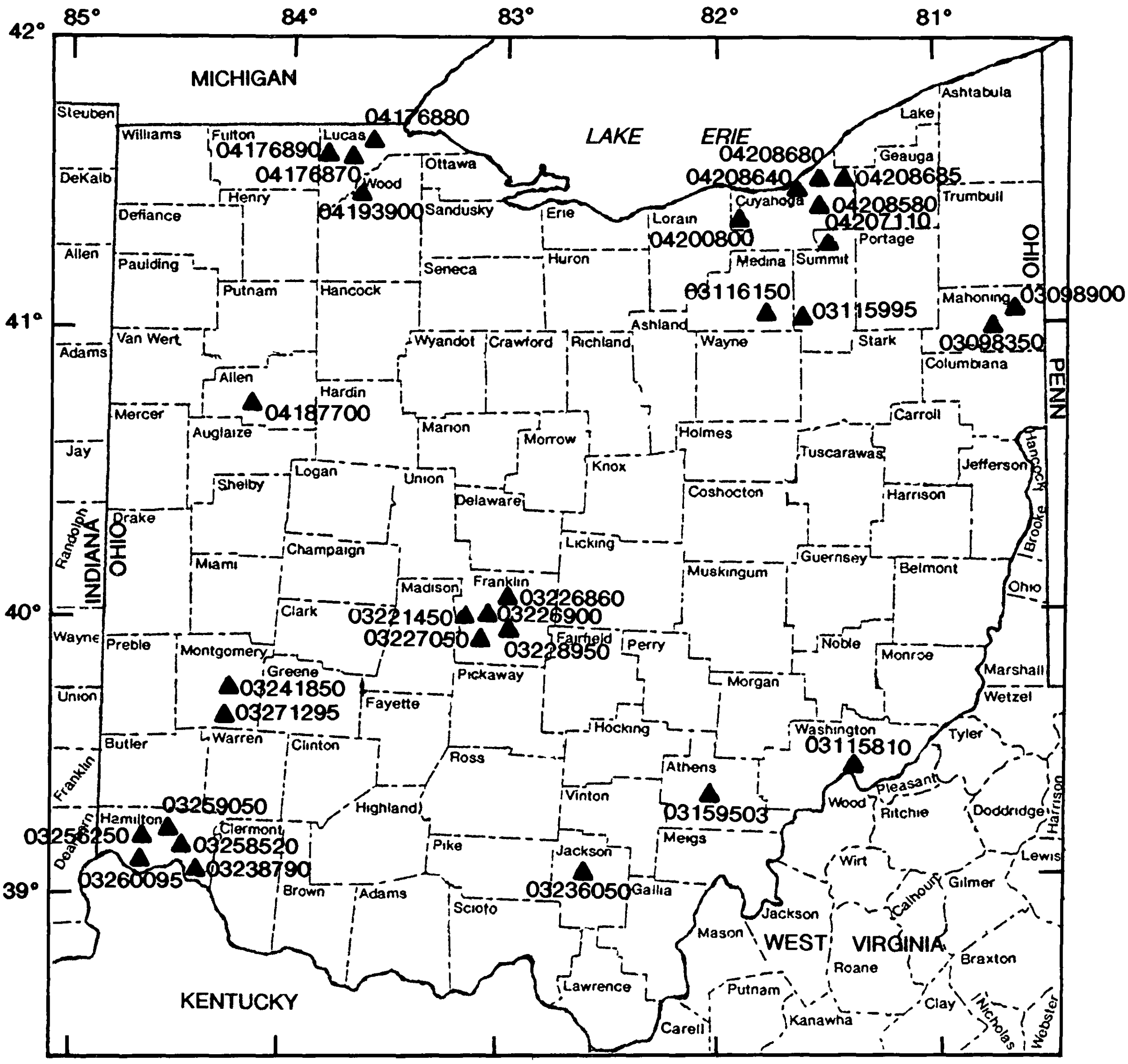
A technique exists for estimating flood hydrographs in which estimated peak discharge and estimated basin lagtime are applied to a dimensionless hydrograph The technique has been successfully applied on a national scale (Stricker and Sauer, 1982) as well as in several statewide studies (Inman, 1987, Robbins, 1986, Sherwood, 1986) and a rural volume study in Ohio (Sherwood, 1993) For this study, the development of a method to estimate flood hydrographs consisted of (1) the use of equations developed as part of this study to estimate peak discharge, (2) the development and use of an equation to estimate basin lagtime for small urban streams, and (3) the verification of a previously developed dimensionless hydrograph for use on small urban streams in Ohio

Development of a method to estimate flood volumes as a function of duration and recurrence interval, which was initially proposed for a study of 32 small rural streams in Ohio (Sherwood, 1993), was expanded to include data from 30 small urban Ohio streams Streamflow data for the 62 small basins were used in this study to develop multiple-regression equations for estimating flood volumes for specific durations and recurrence intervals Six durations (1, 2, 4, 8, 16, and 32 hours) and six recurrence intervals (2, 5, 10, 25, 50, and 100 years) were analyzed, and 36 equations for estimating volume-duration-frequency relations were developed

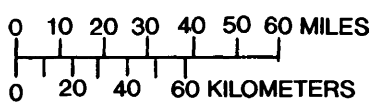
By applying these equations to a design situation in which storage is an important element of the design criteria, the volume of inflow for several durations may be estimated to develop a curve relating inflow volume to duration A theoretical maximum-volume hydrograph based on the volume-duration data may be constructed This hydrograph may be used to develop a relation between inflow volume and time These data can then be used with a volume-elevation curve for the detention basin and an outflow-elevation curve for the outlet to develop a curve relating outflow volume to time. The outflow volume curve can then be subtracted from the inflow volume curve to yield a curve showing the relation between detention-storage volume and time This curve will show the maximum detention storage that might be expected for the specific outlet size, detention-basin size, and the estimated flood characteristics Maximum detention storage calculations for various combinations of outlet size and detention basin size will aid in optimization of the overall design in terms of safety, cost, and efficiency.

DATA COLLECTION

Rainfall and streamflow data were collected at 5-minute intervals at 30 small urban basins for periods ranging from 5 to 8 years (fig 1, table 1) These data were used to calibrate a rainfall-runoff model for each site Sites were chosen in basins where no change in



Base map from U S Geological Survey
United States 1:2 500,000, 1972



EXPLANATION

▲ 03098900 U S GEOLOGICAL SURVEY RAINFALL-RUNOFF STATION AND NUMBER

Figure 1. Approximate locations of urban rainfall-runoff stations (See table 1 for geographical coordinates)

Table 1. Station numbers, station names, latitudes, and longitudes of 30 urban study sites in Ohio

Station number	Station name	Latitude	Longitude
03258520	Amberly Ditch near Cincinnati	39°11'31"	84°25'44"
03238790	Anderson Ditch at Cincinnati	39°04'14"	84°22'51"
03098900	Bunn Brook at Struthers	41°03'05"	80°36'28"
03098350	Charles Ditch at Boardman	41°00'43"	80°39'44"
03236050	Coalton Ditch at Coalton	39°06'36"	82°36'44"
03228950	Dawnlight Ditch at Columbus	40°00'51"	82°56'46"
03260095	Delhi Ditch near Cincinnati	39°05'48"	84°37'23"
04208640	Dugway Brook at Cleveland Heights	41°30'35"	81°34'06"
04208680	Euclid Creek Tributary at Lyndhurst	41°31'52"	81°30'14"
03221450	Fishinger Creek at Upper Arlington	40°01'48"	83°05'12"
03226900	Fishinger Road Creek at Upper Arlington	40°01'25"	83°02'40"
03241850	Gentile Ditch at Kettering	39°42'47"	84°08'56"
04200800	Glen Park Creek at Bay Village	41°29'09"	81°54'46"
04193900	Grassy Creek at Perrysburg	41°33'20"	83°36'45"
03159503	Home Ditch at Athens	39°20'06"	82°04'43"
04176870	Ketchum Ditch at Toledo	41°42'39"	83°35'45"
04208685	Mall Run at Richmond Heights	41°32'35"	81°29'54"
03227050	Norman Ditch at Columbus	39°59'35"	83°02'02"
04208580	North Fork Doan Brook at Shaker Heights	41°28'57"	81°32'34"
03116150	Orchard Run at Wadsworth	41°01'52"	81°44'03"
04187700	Pike Run at Lima	40°46'06"	84°06'57"
03115810	Rand Run at Marietta	39°24'48"	81°25'44"
03226860	Rush Run at Worthington	40°05'41"	82°59'56"
04176880	Silver Creek at Toledo	41°42'58"	83°35'08"
03256250	Springfield Ditch near Cincinnati	39°13'48"	84°31'16"
03115995	Sweet Henri Ditch at Norton	41°01'27"	81°38'13"
04176890	Tifft Ditch at Toledo	41°41'55"	83°37'53"
04207110	Tinkers Creek Tributary at Twinsburg	41°19'30"	81°28'47"
03271295	Whipps Ditch near Centerville	39°39'18"	84°10'10"
03259050	Wyoming Ditch at Wyoming	39°14'00"	84°29'26"

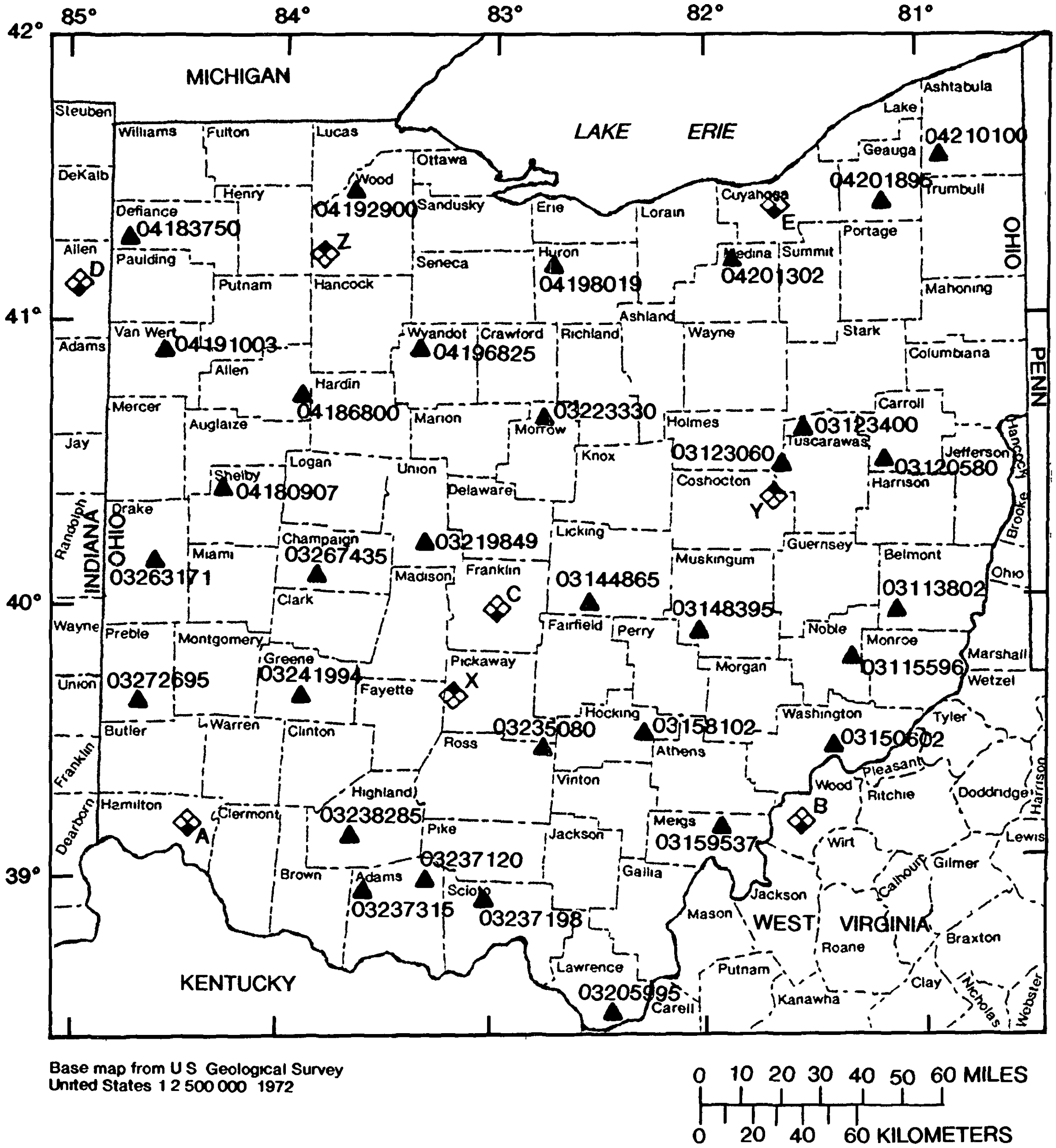
the level of urban development was anticipated for the study period Rainfall and runoff data and calibrated rainfall-runoff models from a concurrent rural volume study were available for 32 rural sites (fig 2, table 2)

All data are stored in the USGS WATSTORE computer data base (National Water Data Storage and Retrieval System) (Hutchinson, 1975)

Synthesized volume data from all 62 rural and urban sites were used for the volume-duration-frequency analysis. Flood volumes generally are not as affected by urbanization as are peak discharges, basin lagtimes, and shapes of flood hydrographs The rates of runoff may be greatly increased due to urbanization because of the effect of decreased roughness on overland and in-channel flow velocities The volumes of runoff also may be increased, but generally to a lesser extent than the increase in rates of runoff The increase in volumes of runoff is a result of increased impervious areas (decreased infiltration) that coincides with urbanization The effects of urbanization on flood volumes is diminished for large floods of long duration For large floods of long duration, soils become saturated (reducing infiltration rates), minimizing the relative influence of impervious areas on flood volumes

Consequently, it was considered reasonable to merge the synthesized volume data from all 62 rural and urban sites for the volume-duration-frequency multiple-regression analyses with an urbanization indicator variable to account for the effects of urbanization on runoff volumes of short duration Because of the significant effects of urbanization on the rates of runoff, however, data from only the 30 urban sites were used in the peak-frequency analyses, basin lagtime analyses, and dimensionless hydrograph verification The following section describes the data-collection methods for the 30 urban study sites The data-collection methods for the 32 rural study sites are very similar and are described in Sherwood (1993)

All streamflow-gaging stations were located at culvert sites where reliable theoretical culvert ratings could be established Stage at each site was sensed by a float-counterweight mechanism in a stilling well and was recorded by a digital recorder The stilling well was positioned at the upstream end of the culvert Data from a crest-stage gage mounted at the downstream end of the culvert was used to verify that there was no backwater at the culvert outlet Stage recorders were installed downstream of culverts at 5 of the 30 sites because of the occurrence of backwater Stage-discharge relations were developed for each site by use of procedures outlined by Bodhaine (1968), in which discharges for a full range of stages are computed indirectly by application of continuity equations and energy equations. Discharge measurements were made by means of a current meter in order to better define the stage-discharge relations at low to medium discharges Measurements were made at high flows whenever possible to provide data required to calibrate



EXPLANATION

▲ 04192900 U S GEOLOGICAL SURVEY RAINFALL-RUNOFF STATION AND NUMBER

C NATIONAL WEATHER SERVICE LONG-TERM RAINFALL STATION AND IDENTIFIER

X NATIONAL WEATHER SERVICE EVAPORATION STATION AND IDENTIFIER

Figure 2. Approximate locations of rural rainfall-runoff stations, long-term rainfall stations, and evaporation stations (See tables 2, 4, and 5 for cross-reference to station numbers and identifiers)

and verify the stage-discharge relations for medium to high flows

Rainfall was recorded at each site by a second digital recorder housed in a steel shelter with a 50-square-inch rainfall collector on top. The shelter was mounted on a 3-inch-diameter aluminum float well that served as a reservoir for the collected rain. A drain tube inside the shelter conveyed rain from the

Table 2. Station numbers, station names, latitudes, and longitudes of 32 rural study sites in Ohio

Station number	Station name	Latitude	Longitude
03115596	Barnes Run at Summerfield	39°47'18"	81°21'08"
04196825	Browns Run near Crawford	40°53'13"	83°20'15"
03235080	Bull Creek near Adelphi	39°27'11"	82°46'46"
04180907	Carter Creek near New Bremen	40°26'16"	84°19'43"
03123060	Cattail Creek at Baltic	40°27'12"	81°42'01"
03113802	Chestnut Creek near Barnesville	39°56'50"	81°09'25"
03148395	Claypit Creek near Roseville	39°50'28"	82°04'15"
04201302	Delwood Run at Valley City	41°14'15"	81°55'18"
03237198	Duncan Hollow Creek near McDermott	38°52'29"	83°03'37"
03123400	Dundee Creek at Dundee	40°35'35"	81°36'13"
03237315	Elk Fork at Winchester	38°56'49"	83°37'21"
03159537	Elk Run near Alfred	39°09'41"	81°57'47"
03120580	Falling Branch at Sherrodsville	40°30'28"	81°14'25"
04201895	Fire Run at Auburn Corners	41°23'36"	81°12'56"
03263171	Harte Run near Greenville	40°08'41"	84°36'41"
04210100	Hoskins Creek at Hartsgrove	41°36'00"	80°57'12"
04186800	King Run near Harrod	40°43'56"	83°53'47"
03267435	Kitty Creek at Terre Haute	40°03'09"	83°52'57"
03223330	March Run near West Point	40°37'55"	82°45'56"
04183750	Racetrack Run at Hicksville	41°18'58"	84°46'00"
04192900	Reitz Run at Waterville	41°29'50"	83°42'35"
04198019	Sandhill Creek near Monroeville	41°12'13"	82°42'56"
03205995	Sandusky Creek near Burlington	38°25'03"	82°30'36"
03150602	Second Creek near Marietta	39°27'36"	81°26'24"
03144865	Slim Creek at Kirkersville	39°56'51"	82°36'13"
03237120	Stone Branch near Peebles	38°57'03"	83°22'29"
04191003	Stripe Creek near Van Wert	40°54'29"	84°33'43"
03238285	Sugar Run near New Market	39°06'30"	83°40'36"
03219849	Tombstone Creek near Marysville	40°12'42"	83°18'15"
03272695	Trippetts Branch at Camden	39°38'03"	84°39'08"
03241994	Twist Run at Xenia	39°39'53"	83°56'00"
03158102	Wolfkiln Run at Haydenville	39°28'35"	82°18'51"

collector to the float well. The rain gage was installed at the site of the streamflow-gaging station if the rainfall would not be intercepted by surrounding trees. Otherwise, the rain gage was installed at an unobstructed, accessible location elsewhere within the basin. A typical rainfall-runoff data-collection station is shown in figure 3.

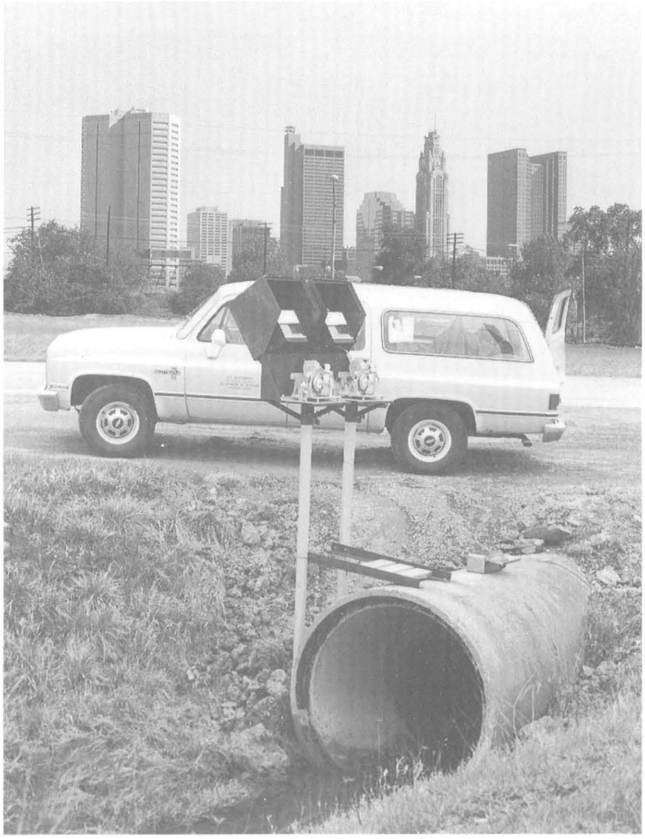


Figure 3. Typical rainfall-runoff data-collection station in Ohio.

Total daily rainfall data were recorded for all days, and 5-minute rainfall and discharge data were recorded for all flood events. Daily rainfall data from a nearby National Weather Service rainfall station were substituted during winter periods and other periods when the recorder was not operational. These substitutions were necessary because the rainfall-runoff model requires continuous daily rainfall data in order to keep an accounting of soil moisture between storm events.

Data were not collected during the winter because the rainfall-runoff model used is not capable of simulating snowmelt runoff. This limitation was not considered significant because most of the major storms that produce large floods on small streams occur during the spring, summer, and autumn in Ohio.

Daily pan evaporation, long-term rainfall for selected storm periods, and long-term daily rainfall also are required for model calibration and long-term (66–87 years) synthesis. These data were obtained from eight National Weather Service stations (fig. 2).

ANALYSIS OF PEAK DISCHARGES AND FLOOD VOLUMES AT STREAMFLOW-GAGING STATIONS

The following sections on model calibration, hydrograph synthesis, peak-frequency analysis, and volume-duration-frequency analysis refer to and briefly describe several computer programs Documentation on the operation of the programs is contained in a user's guide by Carrigan and others (1977).

Calibration of a Rainfall-Runoff Model

Calibrated rainfall-runoff models frequently are 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 problems associated with changing levels of urbanization

The USGS rainfall-runoff model (computer program A634) used for this study was developed by Dawdy and others (1972) and was enhanced by Carrigan (1973), Boning (1974), and Carrigan and others (1977) Model A634 was selected because it is reliable and is less costly and time-consuming in terms of data required and model calibration than most other rainfall-runoff models Input data required for model calibration are daily rainfall, daily evaporation, unit rainfall, and unit discharge (The term "unit data" is used by the USGS to refer to data with a shorter-than-one-day record interval, such as 5 minute, 30 minute, or 3 hour) The hydrologic processes of antecedent soil moisture, infiltration, and surface-runoff routing (table 3) are simulated on the basis of ten model parameters The process of adjusting the parameter values in order to achieve a good fit of simulated hydrographs to observed hydrographs is called calibration

The antecedent soil-moisture accounting component of the model employs four parameters (BMSM, EVC, RR, DRN) and uses daily rainfall and daily evaporation data to simulate the redistribution of moisture in the soil column and evapotranspiration from the soil The infiltration component employs three parameters (PSP, KSAT, RGF) and uses 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

Table 3. 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

uses the Clark unit-hydrograph method to transform the rainfall excess into the outflow hydrograph

Maximum and minimum values were set for each of the 10 parameters Then, within these ranges of values, the parameters were optimized by use of an automatic trial-and-error optimization routine based on a method devised by Rosenbrock (1960)

The model was calibrated for each site in three steps In the first step, the parameters controlling simulated volume (BMSM, EVC, RR, DRN, PSP, KSAT, RGF) were optimized, while the values of the parameters controlling hydrograph shape (KSW, TC, TP/TC) were held fixed In step two, the shape parameters were optimized, while the volume parameters were held fixed In step three, the parameters optimized in step one were readjusted to improve fit of simulated peaks to observed peaks All events were used in the initial calibration

After initial calibration, selected rainfall-runoff events were excluded from further calibrations on the basis of the following criteria

1. Many small events were excluded from model calibration to achieve a more even distribution of small

and large events This was accomplished by excluding most events below a specified minimum peak-discharge threshold. Inclusion of too many small events would give too much weight to small events in the calibration process. This was not desirable, because the calibrated models would be used to synthesize relatively large events

2 Uniform distribution of rainfall over the basin is a major assumption of the model Any discharge events exhibiting an obviously unrepresentative response to 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 obstructed during the event
4 Events were excluded if obvious data-collection problems occurred (such as snowmelt, plugged rainfall collector, or recorder malfunction)

Model parameters values were systematically adjusted until a good fit of simulated to observed hydrographs was achieved

About one-third of the events used for calibration were caused by frontal storms rather than by thunderstorms The frontal-storm-based events generally occurred in early spring or mid-to-late autumn and were generally characterized by better agreement between simulated and observed hydrographs than for thunderstorm-based events The improved agreement probably is a result of the more uniform distribution (both spatial and temporal) of rainfall generally associated with frontal storms Poorer agreement between simulated and observed hydrographs generally was associated with the thunderstorm events, although no bias was indicated for either the frontal-storm or thunderstorm events The final values of parameters used in the calibrated models should permit accurate simulations of runoff caused by rain falling on unfrozen ground

Hydrograph Synthesis

Discharge hydrographs were synthesized for each site by use of the USGS synthesis model (computer program E784, Carrigan and others, 1977) The model combines the calibrated parameter values from the rainfall-runoff model with long-term rainfall and evaporation records to generate a long-term record of synthetic event hydrographs Data from the closest long-term rainfall and evaporation stations for each site were used to synthesize the long-term hydrograph data

Rainfall data were selected from five long-term rainfall stations operated by the National Weather Service (fig. 2). USGS computer program G159 was used to select the 5-minute rainfall data to be used in the long-term synthesis This program scans the daily rainfall records and selects, for each year, up to five of the largest rainfall events that have 1- to 2-day rainfall totals greater than 1 inch An average of three events were selected per year. The daily rainfall data and selected 5-minute rainfall data are used as input for the model

Because of differences in rainfall characteristics between the study sites and the long-term rainfall sites, an adjustment of both the daily and 5-minute rainfall data was considered necessary. Rainfall values at the long-term site were adjusted by multiplying them by the ratio of average annual rainfall at the study site to that of the long-term rainfall site Average rainfall at the study sites was determined from an isohyetal map (Harstine, 1991) based on 50 years (1931–80) of rainfall data from 205 National Weather Service stations Average annual rainfall of the long-term rainfall sites for the 1931–80 period was computed directly from the daily rainfall used for synthesis. The periods of record for each of the five long-term rainfall stations and the number of rainfall events used for hydrograph synthesis are listed in table 4

Data were available from three daily-evaporation data stations operated by the National Weather Service (fig. 2). Ten years of observed record at each site were used to generate an 85-year synthetic record by use of computer program H266 The program averages the 10 daily-evaporation values for each day of the year for the 10-year period and uses those values for the 85-year synthetic record Informa-

Table 4. National Weather Service rainfall stations used in synthesis of hydrograph data

Station number	Location and identifier (fig 2)	Record		
		Number of years	Period	Number of events
390900084310000	Cincinnati, Ohio (A)	80	1897–1976	247
391600081340001	Parkersburg, W Va (B)	77	1899–1975	218
400000082530001	Columbus, Ohio (C)	81	1897–1977	236
410000085130000	Fort Wayne, Ind (D)	66	1911–76	305
412400081510000	Cleveland, Ohio (E)	87	1890–1976	171

Table 5. National Weather Service evaporation stations used in calibration of the rainfall-runoff models and in synthesis of hydrograph data

Station number	Location and identifier (fig 2)	Observed record: Number of years	Observed record: Period	Synthetic record: Number of years	Synthetic record: Period
393800083130000	Deer Creek Lake, Ohio (X)	10	1975–84	85	1890–1974
402200081480000	Coshocton, Ohio (Y)	10	1975–84	85	1890–1974
411300083460000	Hoytville, Ohio (Z)	10	1975–84	85	1890–1974

tion on the periods of record for the daily evaporation sites is summarized in table 5

Peak-Frequency Analysis

The USGS synthesis program E784 was used to analyze annual peak discharges as a function of recurrence interval For each station, the program scans the long-term synthetic-hydrograph (discharge) data, selecting the highest discharge for each water year (A water year is the 12-month period, October 1 through September 30 and is designated by the calendar year in which it ends) The logarithms of the annual peak discharges are then fit by a Pearson Type III frequency distribution

The Pearson Type III frequency analyses were performed as recommended by the Interagency Advisory Committee on Water Data (1982) The skew coefficient used for each site was computed directly from the synthesized data The regional skew map provided by the Committee was not used because it was developed from rural data and may not represent skew coefficients of urban data

Previous investigators have shown that variance in synthetic annual flood data tends to be less than that in observed annual flood data (Lichty and Liscum, 1978, Thomas, 1982) This reduction in variance appears to be at least partially due to a smoothing effect of the rainfall-runoff model The reduction in variance (and, consequently, in standard deviation) of annual flood peaks results in a flattening of the flood-frequency curve for synthetic data, thus, flood estimates for long recurrence intervals (for example, Q_{100}) can be considerably lower than estimates based on observed data At the same time, the flood estimates for short recurrence intervals (for example, Q_2) can be relatively unaffected

Several techniques have been applied to compensate for the bias caused by this reduction in variance Lichty and Liscum (1978) used a bias-adjustment factor, which is the average ratio of the observed to synthetic flood estimates, for the 98 sites in their study for which synthetic and observed data were available The bias-adjustment factors, ranging from 0.98 for the 2-year flood to 1 29 for the 100-year flood, were multiplied by the synthetic flood-frequency data to remove the bias and compute an estimated observed flood-frequency curve with increased discharge at the higher recurrence intervals. Inman (1988) used a technique described by Kirby (1975) whereby the standard deviation of the synthetic annual flood data is divided by the magnitude of a coefficient of correlation between observed and simulated peak discharges A new frequency curve was then computed by use of the adjusted standard deviation and the original mean and skew coefficient Adjusting the frequency curves in this manner increases discharges at higher recurrence intervals.

In Ohio, it was not possible to compute bias-adjustment factors as Lichty and Liscum (1978) did because record lengths (5–8 years) for sites with synthetic data were too short to compute corresponding observed flood-frequency curves for which a minimum of 10 years of record is needed (Interagency Advisory Committee on Water Data, 1982) Also, Kirby's method was not usable in Ohio because there appeared to be little relation between (1) the coefficients of correlation between simulated and observed peak discharges and (2) the standard deviations of simulated and observed peak discharges in the final calibration run

For this study, a method was needed to compensate for reduction in variance of synthetic flood data To accomplish this, an adjustment factor was computed as the ratio of the mean of the coefficients of variation (standard deviations divided by the means) of the logarithms of the annual-peak discharges collected at 97 rural sites having observed data to the mean of the corresponding coefficients of variation of the 32 rural study sites from this study with synthetic data

The range in drainage area for the 32 rural sites with synthetic data is 0 13 to 6 45 square miles, and the average equivalent years of record for the 32 sites is 21 years for the 100-year flood estimate (The average equivalent years of record represents an estimate of the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the synthetic estimate and is computed by use of a method described by Hardison, 1971) The mean coefficient of variation of the logarithms of synthetic annual peak discharges for the 32 sites is 0 146

The 97 rural sites for which observed annual-peak data are available were selected from a data base of 275 rural, unregulated streams in Ohio and adjacent states The 97 sites were chosen to have drainage areas between 0 13 and 6 45 square miles in order to make the synthetic and observed data comparable The average length of systematic record for the 97 sites is 20 5 years The mean coefficient of variation of the logarithms of observed annual peak discharges for the 97 sites is 0 173

The ratio of the mean coefficients of variation for the two data sets is 1 18 (0 173/0 146) The standard deviations of the logarithms of the synthetic annual peak discharges for the 30 urban sites in this study were multiplied by an adjustment factor of 1 18 Adjusted flood-frequency curves were then computed by use of the adjusted standard deviations and the original means and skew coefficients The ratio of the coefficients of variation (1 18) of the two data sets was used as an adjustment factor instead of the ratio of the standard deviations (1 20) to minimize scale effects Comparable standard deviation ratios of 1 23 and 1 25 were computed for data reported by Thomas (1982) and Lichty and Liscum (1978), respectively, for which observed and synthetic data were available The study by Thomas (1982) was based on data from 50 small rural streams in Oklahoma The study by Lichty and Liscum (1978) was based on data from 98 small rural streams in Missouri, Illinois, Tennessee, Mississippi, Alabama, and Georgia

The synthetic-flood-frequency statistics for the 30 urban sites of this study were not incorporated in the computation of the standard-deviation adjustment factor for Ohio because some reductions in variance may be due to urbanization factors Because the standard-deviation adjustment factor is based on rural flood statistics only, any reduction in variance due to urbanization factors will be retained in the adjusted flood-frequency curve.

The adjusted synthetic peak-frequency data for the 30 urban sites are summarized in table 6. The ratios of the mean adjusted *T*-year discharges to the

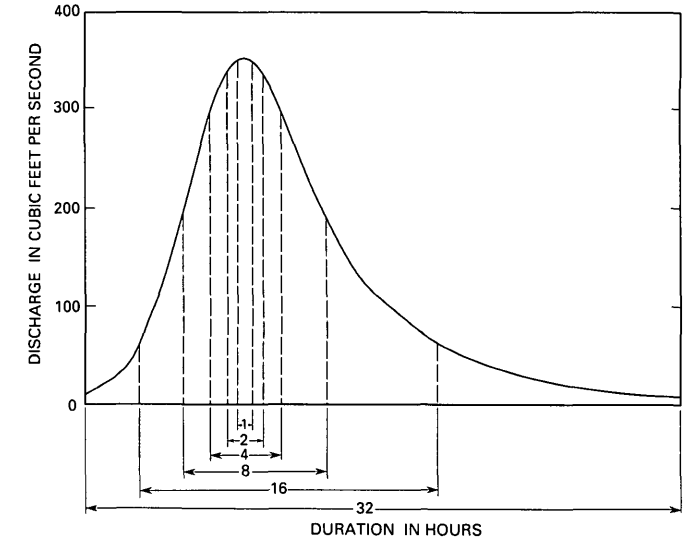


Figure 4. Selection of runoff data for computation of volume for each of six durations (1, 2, 4, 8, 16, and 32 hours)

mean unadjusted *T*-year discharges for the following recurrence intervals are

Q_2 - 1 01
Q_5 - 1 10
Q_{10} - 1 15
Q_{25} - 1 20
Q_{50} - 1 23
Q_{100} - 1 26,

illustrating that the standard-deviation adjustment factor has little effect on the 2-year flood estimate but increases the 100-year flood estimate by about 26 percent on average The ratios listed above are comparable to the bias adjustment factors of 0 98, 1 19, and 1 29 reported by Lichty and Liscum (1978) for the 2-, 25-, and 100-year flood discharges, respectively

Volume-Duration-Frequency Analysis

The USGS synthesis program E784 also was used to analyze flood volumes of the 62 rural and urban study sites as a function of duration and frequency The program was modified to scan the long-term synthetic-hydrograph (discharge) data, and compute the largest runoff volume for each of six durations (1, 2, 4, 8, 16, and 32 hours) for each water year

The volume selection and computation procedure for a single event is illustrated in figure 4. This procedure is performed on all the events for each year, and the annual maximums determined for each duration are used in the volume-frequency analysis Usually, the maximum volumes for all six durations are

Table 6. Explanatory-variable values and peak-frequency data used in the peak-frequency multiple-regression analysis

[*A*, drainage area (in square miles), *P*, precipitation (in inches), *BDF*, basin-development factor (scale from 0 to 12), Peak discharge is in cubic feet per second, Recurrence interval is in years]

Station name	Explanatory variable			Peak discharge for indicated recurrence interval					
	A	*P*	*BDF*	2	5	10	25	50	100
Amberly Ditch	0 14	39 8	9	42	78	104	139	165	192
Anderson Ditch	049	40 1	8	43	74	93	113	126	138
Bunn Brook	51	35 6	8	76	136	182	244	294	346
Charles Ditch	50	35 3	11	173	303	397	521	615	711
Coalton Ditch	50	41 2	0	65	147	220	333	430	538
Dawnlight Ditch	20	36 8	8	63	103	130	164	190	215
Delhi Ditch	16	40 1	10	90	152	194	244	280	314
Dugway Brook	1 42	39 0	12	417	846	1180	1620	1970	2310
Euclid Creek Tributary	1 67	39 4	11	491	799	1000	1240	1420	1580
Fishinger Creek	66	37 2	9	182	337	450	600	715	830
Fishinger Road Creek	45	37 1	11	213	321	391	479	543	606
Gentile Ditch	064	39 2	12	58	86	103	123	136	149
Glen Park Creek	1 21	33 8	4	118	267	392	576	726	886
Grassy Creek	1 81	31 7	6	140	225	285	364	424	485
Home Ditch	24	39 9	3	62	121	160	206	236	262
Ketchum Ditch	84	31 5	10	80	111	131	158	178	198
Mall Run	16	38 5	12	137	227	282	345	387	425
Norman Ditch	60	37 2	10	181	273	333	406	458	509
North Fork Doan Brook	1 18	39 1	10	298	651	936	1330	1650	1970
Orchard Run	43	36 9	11	154	257	325	407	465	520
Pike Run	1 18	35 8	7	194	318	416	559	679	813
Rand Run	33	38 3	4	40	70	92	122	147	172
Rush Run	72	36 6	2	54	91	120	162	197	236
Silver Creek	4 09	31 6	6	167	232	273	325	362	399
Springfield Ditch	26	39 8	9	194	318	400	501	573	643
Sweet Henri Ditch	36	36 7	5	119	216	288	383	456	530
Tifft Ditch	85	31 7	8	99	164	215	290	354	425
Tinkers Creek Tributary	12	40 5	3	43	77	99	126	144	162
Whipps Ditch	2 64	40 3	9	686	1190	1550	2000	2340	2670
Wyoming Ditch	026	39 7	11	24	36	43	52	57	62

computed from the same event. In some cases, however, the short-duration volumes may be selected from a high-peak, short-duration hydrograph, whereas long-duration volumes may be selected from a low-peak, long-duration hydrograph About half of the storms producing annual maximum volumes occurred in the summer The other half of the storms occurred primarily during spring and fall and were evenly divided between spring and fall Only a few storms producing annual maximums occurred during the winter

The logarithms of the annual peak volumes for each duration are then fit by a Pearson Type III frequency distribution The frequency analyses were performed as recommended by the Interagency Advisory Committee on Water Data (1982) The skew coefficient used for each site was computed directly from the synthesized data The regional skew map provided by the Committee was not used because it was developed from rural data and may not represent skew coefficients of urban data

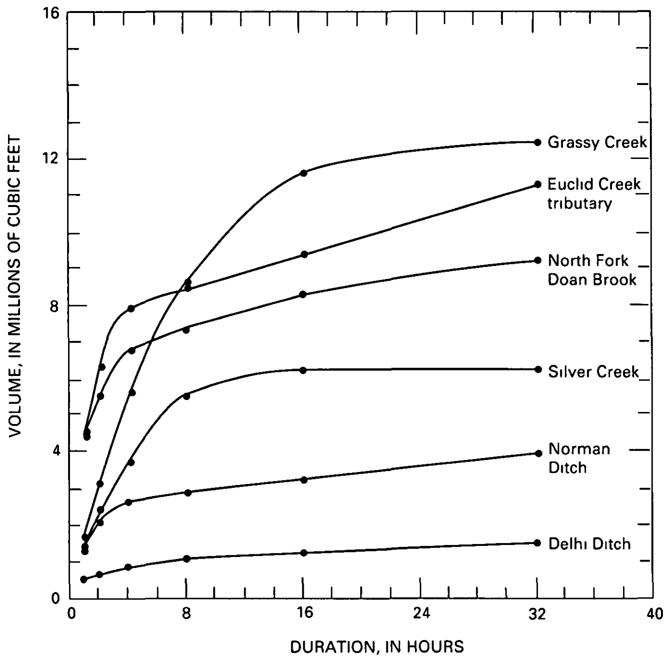


Figure 5. One-hundred-year flood volumes as a function of duration for six study sites in Ohio

It was hypothesized that the standard-deviation adjustment factor applied to the annual-peak-discharge data could be applied to annual-peak-volume data as well, particularly for short durations (1 hour) that are highly correlated with the peak discharges

The standard-deviation adjustment factor of 1 18 used for peak-frequency computations was multiplied by the standard deviations of the logarithms of the annual maximum volumes for each duration for the 62 study sites New volume-duration-frequency curves were then computed by use of the adjusted standard deviations and the original means and skew coefficients. The 100-year volume data are listed in table 7 for all 62 study sites The relation between 100-year volumes and duration for six study sites is shown in figure 5 The symbols on the graphs represent the volume-duration-frequency data computed for each site The lines connecting the symbols are for illustration purposes only.

ESTIMATION OF PEAK-FREQUENCY RELATIONS AT UNGAGED URBAN SITES

It is neither practical nor necessary to collect peak-discharge data at all sites where such information may be required for the design of hydraulic structures Because of the relations among streamflow characteristics and basin characteristics, it is possible to transfer information obtained at gaged sites to ungaged sites (Thomas and Benson, 1970) Methods of transfer range from simple interpolation to complex computer modeling techniques Multiple regression, a method commonly used that has been demonstrated to provide accurate, unbiased, and reproducible results (Newton and Herrin, 1982), was used in this study The method is also relatively easy to apply

Development of Peak-Frequency Equations

Multiple regression is a technique that provides a mathematical equation relating one response variable and two or more explanatory variables The technique also provides a measure of the accuracy of the equation and a measure of the statistical significance of each explanatory variable in the equation In the analysis, several combinations of explanatory variables are tested, the combination that results in a best fit to the observed data is selected, provided that the inclusion of each explanatory variable is hydrologically valid and statistically significant

Peak Discharges as a Function of Basin Characteristics

Peak discharges with recurrence intervals of 2, 5, 10, 25, 50, and 100 years were related to a variety of basin characteristics of the 30 urban sites by use of an equation of the general form

$$UQ_T = a\, A^d\, B^e\, C^f$$

where

UQ_T is urban peak discharge with recurrence interval of T years (response variable),
a is a regression constant,
A,B,C are basin characteristics (explanatory variables), and
d, e, f are regression exponents.

The basin characteristics initially tested in the regression analysis were

A – drainage area
RQ_T – estimated rural peak discharge with recurrence interval of T years
BDF – basin-development factor
IA – impervious area
L – main-channel length
SL – main-channel slope

Table 7 One-hundred-year volumes (dV_{100}) for the 62 study sites in Ohio

Station name	Volume, in millions of cubic feet for indicated duration, in hours					
	1 0	2 0	4 0	8 0	16 0	32 0
Amberly Ditch	0 301	0 402	0 517	0 604	0 646	0 761
Anderson Ditch	256	302	356	405	468	588
Barnes Run	1 18	2 30	4 19	6 64	8 86	9 88
Browns Run	1 60	3 13	5 90	10 3	15 3	20 3
Bull Creek	4 91	9 27	15 5	22 0	25 3	29 6
Bunn Brook	1 03	1 55	2 02	2 16	2 35	2 64
Carter Creek	953	1 87	3 59	6 44	10 1	12 8
Cattail Creek	384	598	939	1 06	1 18	1 26
Charles Ditch	1 91	2 53	3 12	3 24	3 67	4 34
Chestnut Creek	324	603	1 04	1 59	1 97	2 14
Claypit Creek	2 64	5 11	9 33	15 3	20 9	25 0
Coalton Ditch	1 44	2 27	3 06	3 42	3 63	3 89
Dawnlight Ditch	498	652	842	933	1 04	1 20
Delhi Ditch	522	634	828	1 05	1 21	1 50
Delwood Run	611	1 14	2 02	3 17	4 03	4 86
Dugway Brook	5 99	7 54	8 42	9 05	10 3	11 6
Duncan Hollow Creek	927	1 74	2 94	4 28	5 02	6 82
Dundee Creek	1 36	2 29	3 55	5 02	5 67	6 10
Elk Fork	11 6	21 3	36 7	53 7	61 3	84 8
Elk Run	953	1 79	3 00	4 16	4 68	4 99
Euclid Creek Tributary	4 53	6 34	7 92	8 47	9 44	11 3
Falling Branch	559	1 01	1 74	2 49	3 00	3 50
Fire Run	427	797	1 40	2 05	2 50	2 99
Fishinger Creek	1 82	2 34	2 94	3 30	3 64	4 16
Fishinger Road Creek	1 14	1 24	1 47	1 61	1 81	2 15
Gentile Ditch	357	444	538	598	654	874
Glen Park Creek	2 91	4 60	6 33	6 99	7 38	7 66
Grassy Creek	1 67	3 15	5 60	8 65	11 6	12 5
Harte Run	680	1 30	2 35	3 77	5 35	6 27
Home Ditch	746	1 11	1 47	1 95	2 08	2 29
Hoskins Creek	2 21	4 36	8 52	16 2	29 5	48 1
Ketchum Ditch	693	1 31	2 32	3 73	4 70	4 87
King Run	730	1 34	2 23	3 14	3 79	3 96
Kitty Creek	2 06	4 04	7 46	11 7	14 8	17 3
Mall Run	871	1 03	1 22	1 34	1 41	1 67
March Run	394	743	1 21	1 51	1 70	1 98
Norman Ditch	1 44	2 04	2 66	2 88	3 26	3 96
North Fork Doan Brook	4 64	5 55	6 80	7 38	8 29	9 27
Orchard Run	1 34	1 77	2 23	2 32	2 63	2 73
Pike Run	2 14	3 06	3 71	4 62	4 96	5 21
Racetrack Run	567	1 01	1 62	2 20	2 60	2 70
Rand Run	575	1 02	1 61	2 13	2 33	2 46
Reitz Run	555	1 10	2 13	3 90	6 42	8 73
Rush Run	838	1 63	2 89	4 28	5 29	6 23
Sandhill Creek	2 22	4 27	7 63	11 6	14 7	16 8
Sandusky Creek	1 26	2 26	3 98	5 44	6 06	6 42
Second Creek	2 40	4 08	5 97	7 98	9 08	9 32
Silver Creek	1 37	2 44	3 69	5 59	6 24	6 32
Slim Creek	349	617	937	1 14	1 27	1 47
Springfield Ditch	1 26	1 50	1 85	1 96	2 05	2 63
Stone Branch	2 39	3 85	6 23	8 27	10 0	13 7
Stripe Creek	882	1 75	3 40	6 24	10 2	13 5
Sugar Run	3 62	6 41	10 0	13 3	16 2	22 9
Sweet Henri Ditch	1 38	1 87	2 33	2 40	2 73	3 19
Tifft Ditch	1 22	1 87	2 50	3 31	3 48	3 59
Tinkers Creek Tributary	478	677	860	934	1 15	1 39
Tombstone Creek	3 93	7 61	13 9	22 8	31 0	37 9
Trippetts Branch	956	1 63	2 46	2 88	3 39	4 63
Twist Run	1 32	2 33	3 72	5 76	7 81	10 6
Whipps Ditch	7 85	12 3	17 8	22 9	25 7	31 5
Wolfkiln Run	1 05	1 97	3 55	5 76	7 68	8 58
Wyoming Ditch	160	206	251	277	302	397

Table 8. Equations for estimating peak discharges of small urban streams in Ohio

[SER, average standard error of regression (in percent), SEP, average standard error of prediction (in percent), UQ_T, urban peak discharge with average recurrence interval of T years (in cubic feet per second), A, drainage area (in square miles), P, average annual precipitation (in inches), BDF, basin-development factor (on a scale from 0 to 12)]

Equation number	Equation	SER	SEP
(1)	$UQ_2 = 155(A)^{0.68} (P\text{-}30)^{0.50} (13\text{-}BDF)^{-0.50}$	±32.3	±34.3
(2)	$UQ_5 = 200(A)^{0.71} (P\text{-}30)^{0.63} (13\text{-}BDF)^{-0.44}$	±32.8	±34.8
(3)	$UQ_{10} = 228(A)^{0.74} (P\text{-}30)^{0.68} (13\text{-}BDF)^{-0.41}$	±33.7	±36.0
(4)	$UQ_{25} = 265(A)^{0.76} (P\text{-}30)^{0.72} (13\text{-}BDF)^{-0.37}$	±35.0	±37.6
(5)	$UQ_{50} = 293(A)^{0.78} (P\text{-}30)^{0.74} (13\text{-}BDF)^{-0.35}$	±35.9	±38.8
(6)	$UQ_{100} = 321(A)^{0.79} (P\text{-}30)^{0.76} (13\text{-}BDF)^{-0.33}$	±36.9	±40.1

$L/\sqrt{SL}$ – main-channel length divided by the square root of the main-channel slope
LT – basin lagtime
EL – average main-channel-elevation index
P – average annual precipitation
$0.5RF_2$ – 2-year, 0.5-hour rainfall
$2RF_2$ – 2-year, 2-hour rainfall
$3RF_2$ – 2-year, 3-hour rainfall

These basin characteristics were chosen for consideration in this analysis because of their significance in previous studies (Webber and Bartlett, 1977, Sauer and others, 1983, Sherwood, 1986, Koltun and Roberts, 1990) Basin storage (ST) was not tested in the regression analysis because all sites were chosen to have little or no storage (of the 30 study sites, 24 had no storage, of the 6 that had storage, the maximum was 0.20 percent of the total drainage area)

Multiple-regression analyses were performed by use of the Statistical Analysis System (SAS Institute, 1982) A combination of step-forward and step-backward procedures was used to assist in determining which of the explanatory variables should be included in the six regression equations.

The analysis resulted in the six regression equations listed in table 8 The equations can be used to estimate peak discharges of specific recurrence intervals for small urban streams in Ohio The accuracy and limitations associated with the equations are discussed in subsequent parts of this report The average standard error of regression (SER) and average standard error of prediction (SEP) have been computed for each equation and are listed in table 8

The average standard error of regression, in the context of this analysis, is a measure of an average error between synthetic peak discharges and regression-estimated peak discharges for the 30 gaged sites and indicates how well the equations estimate peak discharges for the 30 gaged sites used in the regression analysis The average standard error of prediction, however, is an approximation of the accuracy of the equations for estimating peak discharges at sites not included in the regression analysis It is computed by leaving out 1 site, developing an equation based on the other 29 sites, and computing the residual for the site left out. The process is repeated for each site, and the 30 residuals are squared and summed The sum of the squared residuals, called the PRESS statistic (Montgomery and Peck, 1982), may be computed by various statistical computer programs including the Statistical Analysis System (SAS Institute, 1982)

The standard error of prediction is computed by taking the square root of the PRESS statistic multiplied by γ, where γ is defined as

$$\gamma = \left(\frac{n-1}{n}\right)\left(\frac{n-p-3}{n-p-2}\right)\left(\frac{n+1}{n}\right)\left(\frac{n-2}{n-3}\right)\left(\frac{1}{n}\right)$$

where n is the number of observations, and p is the number of degrees of freedom (Edward J. Gilroy, U.S. Geological Survey, Reston, Va., written commun., 1988)

The values of the three statistically significant explanatory variables (A, P, BDF) are listed in table 6 The variables were transformed to improve the linearity of the relations between the response and explanatory variables and to reduce the standard errors

1 A constant of 30 inches was subtracted from all values of P The minimum value of P for Ohio is about 31 inches
2 BDF was subtracted from 13 In a nationwide urban study, Sauer and others (1983) found that equation

accuracy was improved if BDF was used on a reverse scale (13–BDF) In this study, both BDF and 13–BDF were tested, and 13–BDF yielded the best results

3 The final values of all response (UQ_T) and explanatory variables (A, P–30, 13–BDF) were transformed by taking base 10 logarithms Past experience in hydrologic studies has shown that the linearity of many relations between streamflow characteristics and basin characteristics is improved if the logarithms of each are used (Thomas and Benson, 1970)

All explanatory variables in equations 1 through 6 (table 8) had significance levels equal to or less than 1 percent

Sensitivity Analysis

Errors in measurement or judgment may occur when determining values for the physical and climatic variables (A, P, and BDF) Consequently, a sensitivity analysis was performed to illustrate the effects of errors in these variables on the computations of peak discharges The means of the three explanatory variables for the 30 study sites were calculated to be

A = 0 778 square miles,
BDF = 7 97, and
P = 37 3 inches

These values were substituted into the six regression equations Each explanatory variable was then varied from its mean in 5-percent increments from –50 percent to +50 percent, while the values of the other variables were held constant. The percentage of change in the explanatory variable was then plotted against the percentage of change in the computed peak discharge The results are presented in figure 6 (Because all six plots were similar, only the UQ_5, UQ_{25}, and UQ_{100} plots are shown)

The sensitivity for each explanatory variable is the change in the computed peak discharge as a function of the change in the explanatory variable Computed peak discharges are least sensitive to changes in explanatory variables that plot closest to the horizontal axes in figure 6 Conversely, the computed peak discharges are most sensitive to changes in explanatory variables that plot farthest from the horizontal axes Explanatory variables which plot as straight lines (A and P) indicate that the sensitivity of peak discharge to that variable does not change as the value of that variable changes. Explanatory variables which plot as curved lines (BDF) indicate that the sensitivity of peak discharge to that variable does change as the value of that variable changes

In the case of BDF, peak discharges become increasingly sensitive to changes in BDF as the value of BDF increases Thus, an accurate evaluation of BDF seems to be more critical in the range from 8 to 12 In contrast, the sensitivity of urban peak discharges to changes in A and P remains fairly constant for a given recurrence interval

The sensitivity of peak discharges to changes in BDF decreases for floods with higher recurrence intervals The tendency for BDF to have less effect at higher recurrence intervals can be explained The amount of impervious area (IA), which is closely related to BDF, tends to have less effect on flood characteristics during large floods because infiltration rates are reduced due to saturation In addition, flood peaks on highly developed basins may be somewhat attenuated during large floods because of temporary storage behind culverts, bridges, and storm sewers

Tests for Intercorrelation and Bias

All significant variables were checked for intercorrelation A high degree of intercorrelation between explanatory variables may affect the magnitude and sign of their regression exponents as well as reducing their statistical significance Values of Pearson correlation coefficients may range from +1 0 to –1 0, computed values close to +1 0 or –1 0 indicate a high degree of intercorrelation. The following matrix shows the Pearson correlation coefficients of the base-10 logarithms of the three explanatory variables in the peak-frequency equations

	A	***P*–30**	**13–*BDF***
A	1 00	–0 51	+0 18
P–30		1 00	– 17
13–BDF			1 00

The most highly correlated variables, A and P–30, have a correlation coefficient of –0 51 The Pearson correlation coefficient and other statistical tests for multicollinearity (variance inflation factor and condition number) indicate that the predictive ability of the equations are not appreciably affected by intercorrelation

All equations were checked for parametrical and geographical bias Parametrical bias was tested by plotting the residuals (differences between the synthe-

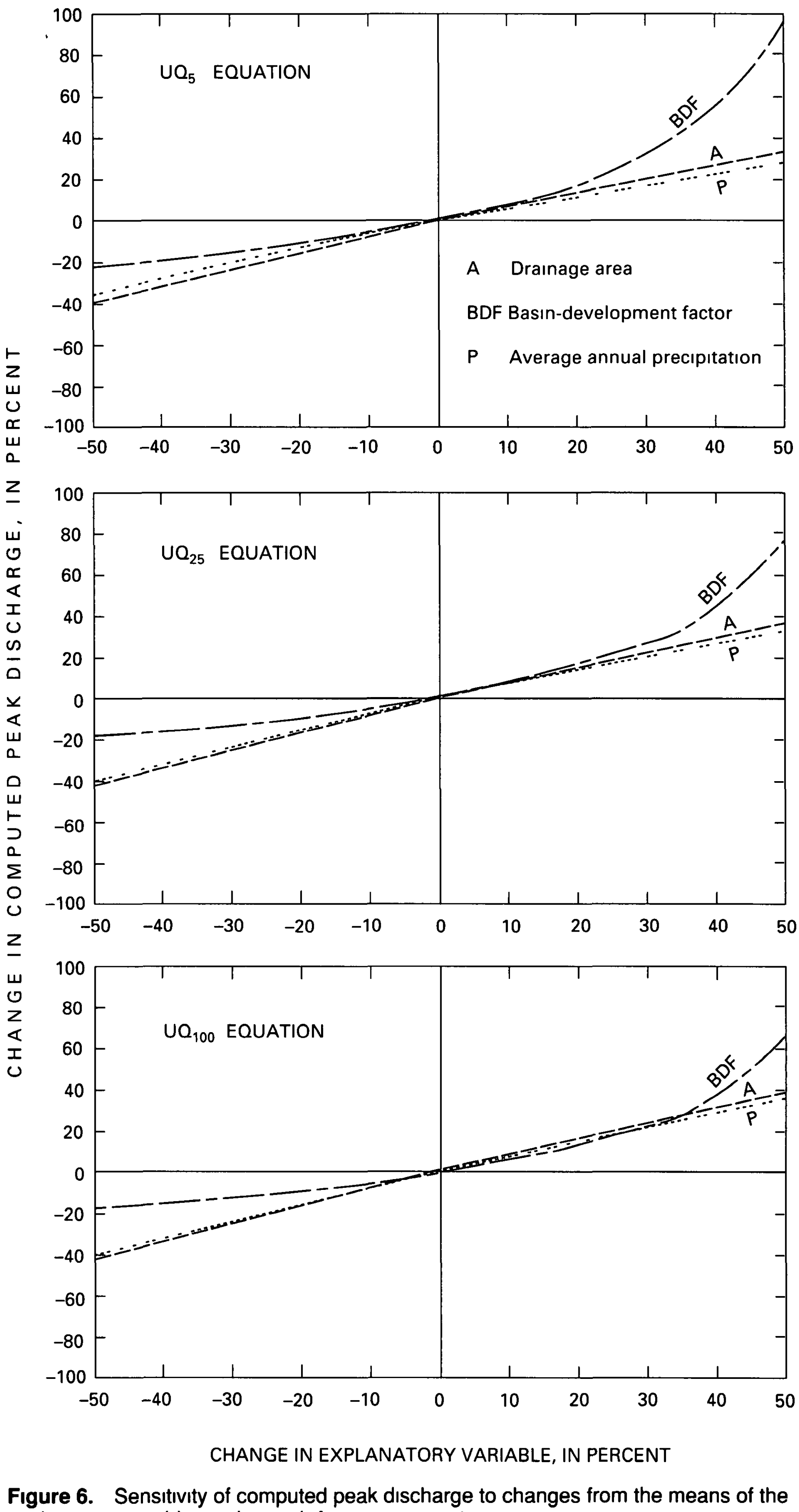


Figure 6. Sensitivity of computed peak discharge to changes from the means of the explanatory variables in the peak-frequency equations

sized and regression estimates) against each of the response and explanatory variables Visual inspection of the plots indicated that the signs and magnitudes of the residuals varied randomly throughout the ranges of the response and explanatory variables, thus indicating no apparent parametrical bias

The relation between residuals and the location of urban development within the basin also was explored because it was hypothesized that development in the upper end of the basin may increase peak discharges more than development in the lower end Five sites have significantly more development in either the upper or lower end of the basin No trends in the residuals were apparent, suggesting that the relative location of urban development within the basin may not affect the peak discharges at these sites

To test for geographical bias, the residuals for each site and recurrence interval were plotted on State maps at the corresponding locations for those sites These plots were then inspected to determine if residuals of a given sign tended to cluster in any city or geographic region of the State No geographical bias was apparent

Application of Peak-Frequency Equations

The six peak-frequency equations provide a means for estimating peak discharges for selected recurrence intervals at ungaged urban sites

Limitations of the Method

The six multiple-regression equations developed for estimating peak-frequency relations are applicable to sites on small urban streams in Ohio whose basin characteristics are approximately within the range of the basin characteristics of the 30 study sites used in the regression analysis The following table shows the ranges of the basin characteristics of the study sites

Basic characteristic	Minimum	Maximum	Unit
A	0 026	4 09	square miles
P	31 5	41 2	inches
BDF	0	12	scale from 0 to 12

Application of the equations to streams having basin characteristics outside of these ranges may result in errors that are considerably greater than those implied by the standard error of prediction

All study sites were chosen to have minimal (less than 1 0 percent of the total drainage area) basin storage The equations are not applicable to streams whose flood characteristics are significantly affected by storage or where upstream culverts or other structures may significantly reduce peak discharges by temporarily storing water behind them

It was assumed in this study that annual-peak discharges of small urban streams in Ohio are caused by rain falling on unfrozen ground Data were collected and analyzed accordingly The equations, therefore, should not be applied to streams where annual peak discharges are likely to be affected by snowmelt or frozen ground

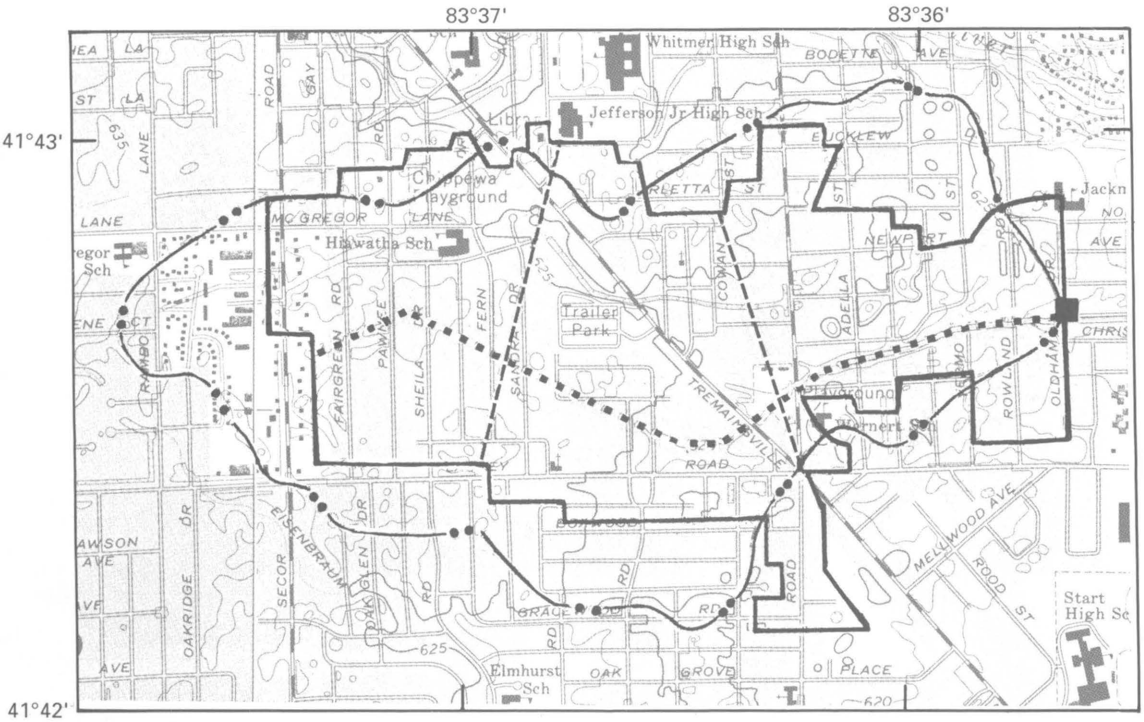
Computation of Basin Characteristics

The values of the three basin characteristics are entered into the appropriate regression equations to compute the peak discharge for the desired recurrence intervals The basin characteristics may be determined as follows

A Drainage area (in square miles)—The drainage area contributing surface runoff to a specified location on a stream, measured in a horizontal plane Computed (by planimeter, digitizer, or grid method) from USGS 7 5-minute topographic quadrangle maps (fig 7) Sewer maps may be necessary to delineate drainage area in urban areas because sewer lines sometimes cross topographic divides

P Average annual precipitation (in inches)—Determined from an isohyetal map, shown in figure 8 and published by the Ohio Department of Natural Resources (Harstine, 1991)

BDF Basin-development factor (on a scale from 0 to 12)—A measure of urban development within the basin The following description of how to determine *BDF* is based upon 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 that are approximately perpendicular to the main channel and principal tributaries (figure 9) Flood-peak travel times for streams within each third should be about equal The subdivisions are generally drawn by eye, as precise measurement is not necessary Four aspects of the drainage system are then evaluated within each third of the basin and each third



Base from U.S. Geological Survey
Toledo, 1:24,000, 1965, photorevised 1980;
Sylvania, 1:24,000, 1965, photorevised 1980

0 — 1 MILE

0 — 1 KILOMETER

EXPLANATION

SEWER MAP BASIN DIVIDE—Used to determine drainage area

TOPOGRAPHIC MAP BASIN DIVIDE

LINES SUBDIVIDING THE BASIN INTO THIRDS FOR DETERMINING THE BASIN-DEVELOPMENT FACTOR

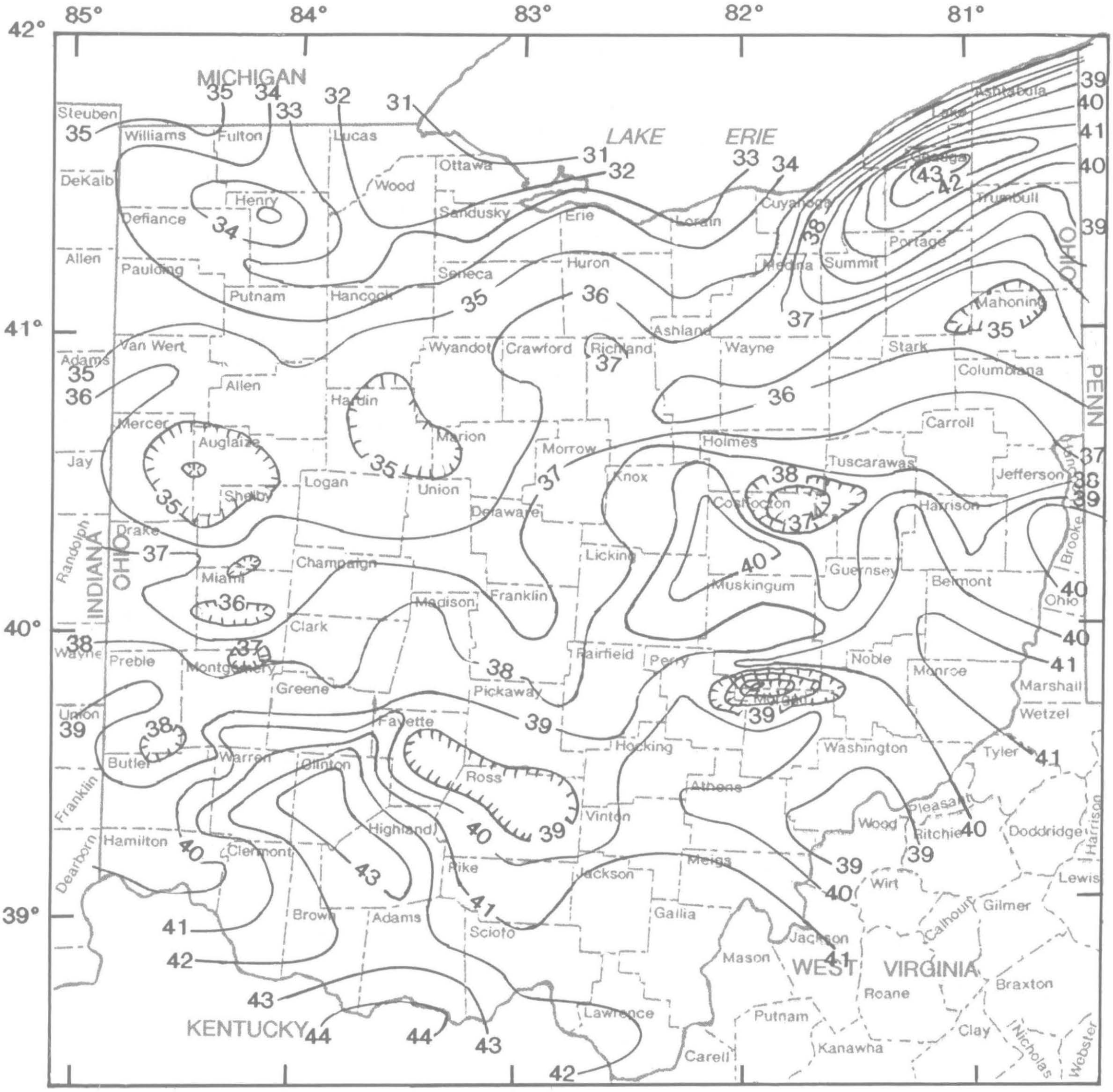
MAIN DRAINAGE CHANNEL

UNGAGED SITE

Figure 7. Ungaged urban stream in Toledo, Ohio.

assigned a code as follows (Sauer and others, 1983):

1. Channel improvements—If channel improvements [in terms of the ability of the channel to transport water] 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 these improvements would qualify for a code of 1. To be considered prevalent, at least 50 percent of the main channels and principal tributaries must be improved to some degree over natural



Base map from U.S. Geological Survey
United States 1:2,500,000, 1972

EXPLANATION

—34— LINE OF EQUAL AVERAGE ANNUAL PRECIPITATION—Hachured lines enclose areas of lesser precipitation. Interval is 1 inch

Figure 8. Average annual precipitation for Ohio for 1931–80 (modified from Harstine, 1991).

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

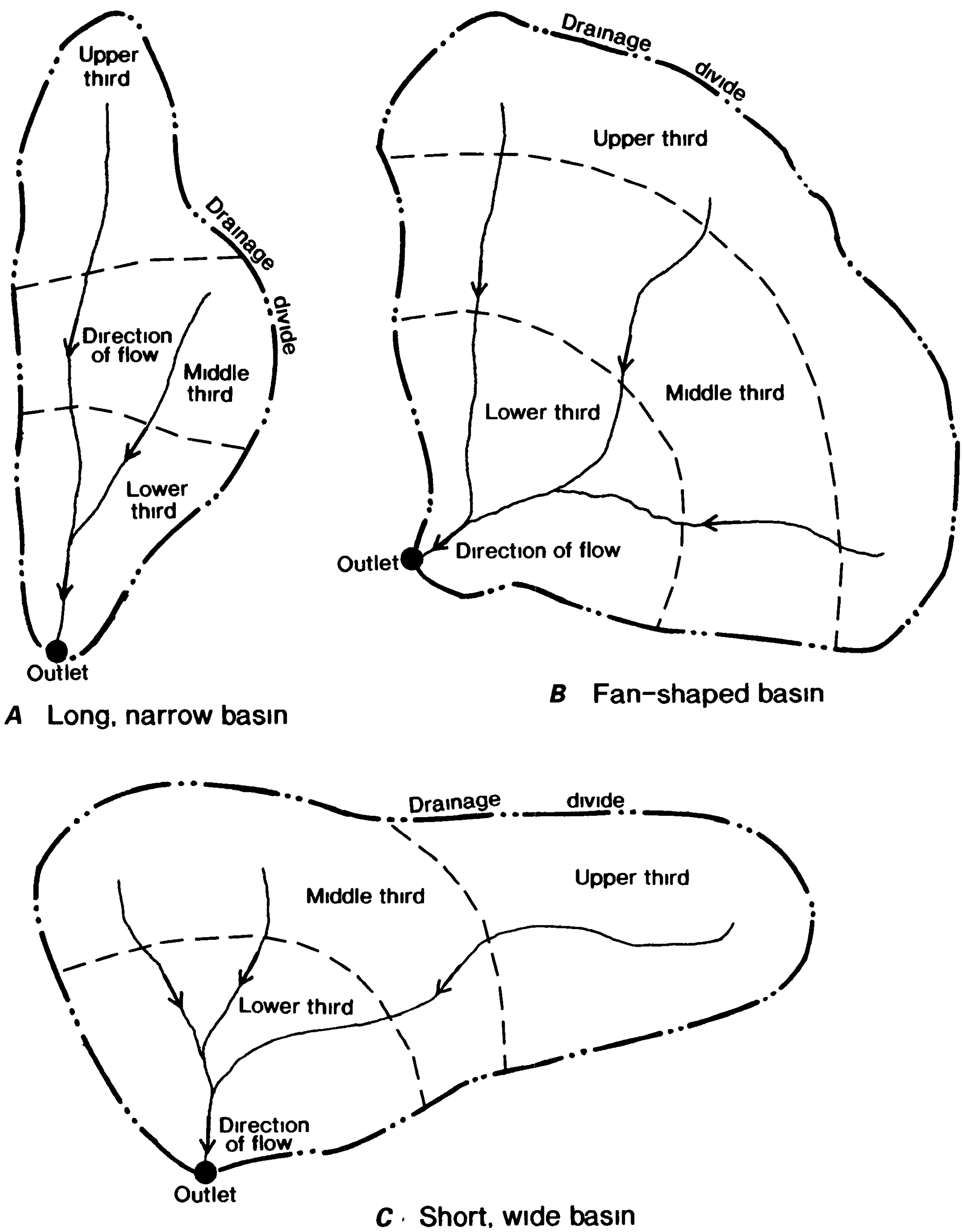


Figure 9. Schematic of typical drainage basin shapes and subdivision into thirds (from Sauer and others, 1983)

the presence of channel improvements as well Therefore, this is an added factor and indicates a more highly developed drainage system.

3. Storm drains (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 also would 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, 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 *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 not urbanized 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

The BDF may be readily estimated for an existing urban basin The 50-percent guideline will usually not be difficult to evaluate because many urban areas tend to use the same design criteria, and therefore have similar drainage aspects, throughout Also, *BDF* may be used to estimate the effects of future development on flood peaks Obviously, full development and maximum urban effects on peaks would occur when *BDF* equals 12 Projections of full or intermediate stages of development can usually be obtained from city engineers

For the convenience of the reader, a field note sheet for *BDF* evaluation is shown in figure 10

Computation of Peak Discharges

The following steps describe the procedure used to estimate peak discharges of small urban streams in Ohio

1 Determine the values of *A*, *P*, and *BDF* as described above
2 Check that the characteristics of the basin meet the criteria described previously in "Limitations of the Method"
3 Select the appropriate equations from table 8 for the desired recurrence interval
4 Substitute the values of *A*, *P*, and *BDF* into the equation
5 Compute the peak discharge

Example Computation of Peak Discharge

Estimate the peak discharges for the 25-year and 100-year floods for an ungaged urban stream in Toledo, Ohio (fig 7)

1 The following basin characteristics are determined

A = 0 89 square miles
P = 31 6 inches
BDF = 9

Irregularity of the drainage-area boundary and nonconformity with the natural basin divide is illustrated in figure 7 The location of the boundary was determined from sewer maps

2 The basin characteristics meet the criteria described in "Limitations of the Method "
3 The appropriate equations to be applied from table 8 are

$$UQ_{25} = 265\,(A)^{0\,76}(P-30)^{0\,72}\,(13-BDF)^{-0\,37}$$
$$UQ_{100} = 321\,(A)^{0\,79}(P-30)^{0\,76}\,(13-BDF)^{-0\,33}$$

4 The basin characteristics are substituted into the equations

$$UQ_{25} = 265(0.89)^{0\,76}(31\,6-30)^{0\,72}(13-9)^{-0\,37}$$
$$UQ_{100} = 321(0\,89)^{0\,79}(31\,6-30)^{0\,76}(13-9)^{-0\,33}$$

5 The estimated peak discharges are

UQ_{25} = 204 cubic feet per second
UQ_{100} = 265 cubic feet per second

ESTIMATION OF FLOOD HYDROGRAPHS AT UNGAGED URBAN SITES

Estimated flood hydrographs provide a means of routing design peak discharges through a hydraulic structure so that outflow peak discharges from the

BASIN-DEVELOPMENT FACTOR

FIELD NOTES

STATION NAME ______________________________

LOCATION ____________________ I D NUMBER ____________________

EVALUATOR ____________________ DATE ____________________

ASPECT	THIRD	CODE	REMARKS
Channel Improvements	Lower		
	Middle		
	Upper		
Channel Linings	Lower		
	Middle		
	Upper		
Storm Sewers	Lower		
	Middle		
	Upper		
Curb & Gutter Streets	Lower		
	Middle		
	Upper		

BDF =

Figure 10. Field note sheet for evaluating basin-development factor (*BDF*)

structure may be estimated A relatively simple technique for estimating flood hydrographs, in which estimated peak discharge for a specific recurrence interval and estimated basin lagtime (*LT*) are applied to a dimensionless hydrograph, has been successfully applied in a national study (Stricker and Sauer, 1982) and also in several statewide studies (Inman, 1987, Robbins, 1986, Sherwood, 1986) and was selected for use in this study Integrating the area under the estimated hydrograph provides a volume estimate associated with the estimated peak discharge

The dimensionless hydrograph is developed by first computing unit hydrographs for many observed flood hydrographs at many sites The unit-hydrograph computation method is by O'Donnell (1960) These unit hydrographs are then reduced to dimensionless terms by dividing each discharge value by the peak discharge and each corresponding time value by the basin lagtime The hydrograph peaks are then aligned and the discharge values are averaged for each 5-minute time increment to produce an average dimensionless hydrograph The dimensionless hydrograph method is described in detail by Inman (1987)

The dimensionless hydrograph is based on streamflow and rainfall data Rainfall data is included in its derivation, but not in its application The method produces a typical (or average) hydrograph with a recurrence interval equal to the recurrence interval of the estimated peak discharge. Removal of rainfall from the application makes the dimensionless hydrograph method simple and easy to apply The effects of rainfall duration on hydrograph duration are indirectly included however, because of the effects of rainfall duration on basin lagtime, which is used in the application

Development of a Hydrograph-Estimation Technique for Ohio

The development of a hydrograph-estimation technique for urban Ohio streams consisted of (1) the use of equations developed as part of this study to estimate peak discharges of urban streams, (2) the development of an equation to estimate basin lagtimes of urban streams, and (3) the verification of a previously developed dimensionless hydrograph for use on small urban streams in Ohio

Estimation of Peak Discharge

Use of the dimensionless hydrograph method for the simulation of flood hydrographs requires a value for peak discharge Most design applications will use a peak-discharge value associated with some specified recurrence interval. However, the method may also be used to fit the dimensionless hydrograph to an actual peak discharge In this case, the method will not reproduce the actual flood hydrograph, nor is it intended to, the simulated hydrograph will simply be an average hydrograph typical of average rainfall and antecedent conditions If the peak discharge is to be estimated, equations 1 through 6 (table 8) are applicable

Estimation of Basin Lagtime

Basin lagtime (*LT*) is generally defined as the time elapsed from the centroid of the rainfall excess (rainfall contributing to direct runoff) to the centroid of the resultant runoff hydrograph When applied to a dimensionless hydrograph, estimated lagtime is used to define the width (time) of the hydrograph, whereas estimated peak discharge is used to define the height (discharge) The average basin lagtime for each of the 30 urban study basins was computed as *KSW*+1/2 *TC*, a relation previously defined by Kraijenhoff van de Leur (1966), where *KSW* and *TC* (table 3) are those parameter values computed in the final model calibrations for each site Average basin lagtimes were then related to the basin characteristics of the 30 urban study sites (table 9) by multiple-regression analysis

The analysis resulted in the following equation

$$LT=1\ 13\ (L/\sqrt{SL})^{0\ 57}\ (13-BDF)^{0\ 46} \qquad (7)$$

where

LT = lagtime (hours),
L = main-channel length (miles),
SL = main-channel slope (feet per mile), and
BDF = basin-development factor (scale from 0 to 12)

The average standard error of regression is ±50 percent, and the average standard error of prediction is ±53 percent Both explanatory variables are statistically significant at the 1-percent level Bias tests indicated no apparent parametrical or geographical bias

A sensitivity analysis was performed to illustrate the effects of errors in *L*, *SL*, and *BDF* on computations of basin lagtime The mean values of *L* (1 52 miles), *SL* (92 8 feet per mile), and *BDF* (7 97) were substituted into the lagtime equation, and each explanatory variable was then varied by 5-percent increments from −50 percent to +50 percent while the values of the other explanatory variables were held constant The percent change in the explanatory variable was then plotted against the percent change in the computed lagtime The results are shown in figure 11.

Computed basin lagtime will be least affected by changes in explanatory variables that plot closest to

Table 9. Values of basin lagtime, main-channel length, main-channel slope, and basin-development factor used in the basin lagtime multiple-regression analysis

[*LT*, basin lagtime (in hours), *L*, main-channel length (in miles), *SL*, main-channel slope (in feet per mile), *BDF*, basin-development factor (on a scale from 0 to 12)]

Station name	*LT*	*L*	*SL*	*BDF*
Amberly Ditch	0 18	0 54	287	9
Anderson Ditch	24	38	333	8
Bunn Brook	1 11	1 60	58 3	8
Charles Ditch	72	1 27	31 5	11
Coalton Ditch	1 34	1 46	110	0
Dawnlight Ditch	59	62	65 0	8
Delhi Ditch	17	74	127	10
Dugway Brook	55	2 82	70 9	12
Euclid Creek Tributary	88	3 18	44 0	11
Fishinger Creek	52	1 41	61 5	9
Fishinger Road Creek	22	1 05	73 7	11
Gentile Ditch	44	30	44 4	12
Glen Park Creek	1 78	1 92	48 6	4
Grassy Creek	4 23	2 72	8 6	6
Home Ditch	1 09	98	68 3	3
Ketchum Ditch	3 62	1 54	13 0	10
Mall Run	32	68	78 5	12
Norman Ditch	83	2 16	46 3	10
North Fork Doan Brook	58	2 10	86 3	10
Orchard Run	53	1 15	116	11
Pike Run	1 03	1 72	24 8	7
Rand Run	1 02	1 08	141	4
Rush Run	4 87	2 50	8 0	2
Silver Creek	1 94	4 50	14 8	6
Springfield Ditch	27	85	117	9
Sweet Henri Ditch	67	1 20	72 2	5
Tifft Ditch	1 15	1 89	19 4	8
Tinkers Creek Tributary	1 16	63	94 9	3
Whipps Ditch	1 29	2 49	58 9	9
Wyoming Ditch	17	18	462	11

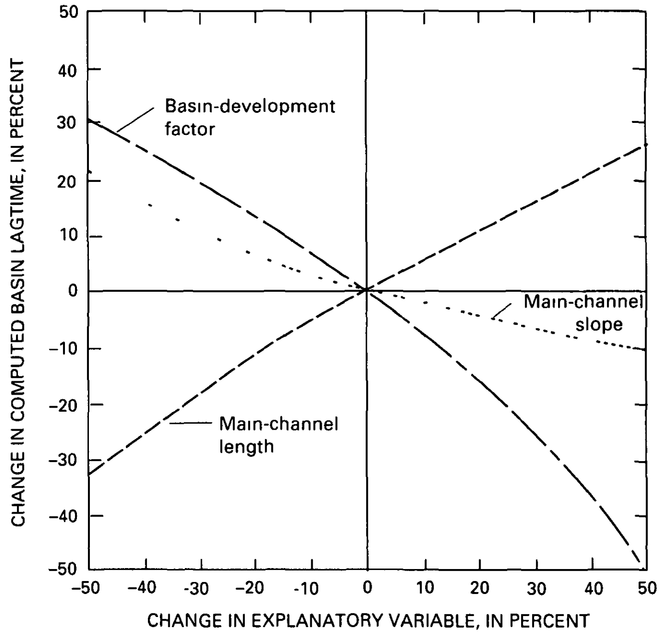


Figure 11. Sensitivity of basin lagtime (*LT*) to changes from the means of the explanatory variables in the basin-lagtime equation

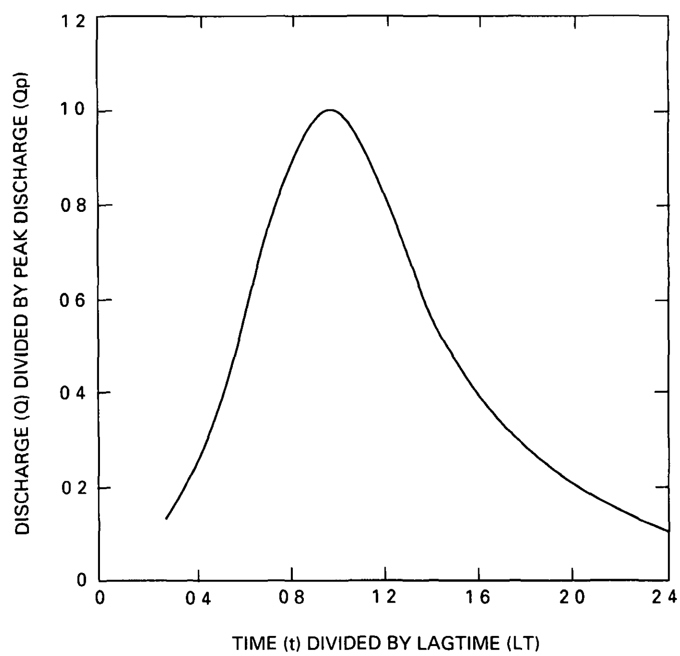


Figure 12. Dimensionless hydrograph (from Inman, 1987)

the horizontal axis in figure 11 Conversely, the computed basin lagtime is most sensitive to changes in explanatory variables that plot farthest from the horizontal axis

Selection and Verification of 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 (Q/Q_p) and the time as the ratio of time to lagtime (t/LT) as shown in figure 12 and table 10 It is developed by averaging typical hydrographs from a variety of basins The hydrographs used in the analysis are single-peak events of average duration Previous investigators have developed several dimensionless hydrographs, most of which are very similar

Table 10. Time and discharge ratios of the dimensionless hydrograph

[From Inman, 1987, *t*, time (in hours), *LT*, lagtime (in hours), *Q*, discharge (in cubic feet per second), and Q_p, peak discharge (in cubic feet per second)]

Time ratio (*t*/*LT*)	Discharge ratio (*Q*/*Qp*)
0 25	0 12
30	16
35	21
40	26
45	33
50	40
55	49
60	58
65	67
70	76
75	84
80	90
85	95
90	98
95	1 00
1 00	99
1 05	96
1 10	92
1 15	86
1 20	80
1 25	74
1 30	68
1 35	62
1 40	56
1 45	51
1 50	47
1 55	43
1 60	39
1 65	36
1 70	33
1 75	30
1 80	28
1 85	26
1 90	24
1 95	22
2 00	20
2 05	19
2 10	17
2 15	16
2 20	15
2 25	14
2 30	13
2 35	12
2 40	11

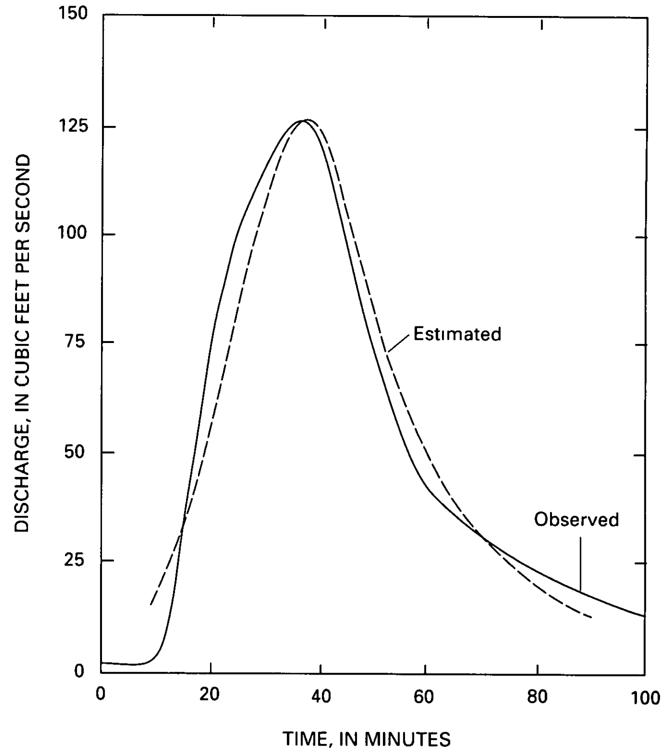


Figure 13. Observed and estimated hydrographs for flood event of May 14, 1983, on Charles Ditch at Boardman, Ohio

A dimensionless hydrograph developed by the USGS for use in Georgia (Inman, 1987) was selected for application in this study for several reasons

1 The basins used in its development were similar in size and land use to the basins used in the Ohio study It was developed from 80 basins (61 rural, 19 urban) all of which had drainage areas less than 20 square miles The dimensionless hydrograph was verified for use on rural and urban streams in the Georgia study
2 The Georgia dimensionless hydrograph has been verified for estimation of flood hydrographs on small rural streams in Ohio (Sherwood, 1993)
3 The Georgia dimensionless hydrograph was verified for use in Tennessee (Robbins 1986) for both urban and rural streams, which further supports its applicability in other humid eastern States

The Georgia dimensionless hydrograph was verified for use in Ohio by applying it to data for 10 of the 30 sites used in this study The 10 sites were selected to be distributed throughout the State and to represent the full range of values of drainage area and basin lagtime encountered in this study Estimated hydrographs were compared with observed hydrographs at each of the 10 sites as illustrated in figure 13 The estimated hydrographs were determined by applying the average station lagtime and peak discharge of the observed hydrograph to the Georgia dimensionless hydrograph The estimated and observed hydrographs compared well at all 10 sites, with no tendency to overestimate or underestimate the widths of the hydrographs The coordinates of the dimensionless hydrograph developed by the Georgia District and verified for use in Ohio are listed in table 10 and plotted in figure 12

Application of the Hydrograph-Estimation Technique

The following sections describe a technique for estimating flood hydrographs for a specified peak discharge The technique is applicable to small urban streams in Ohio in which flood characteristics are not significantly affected by basin storage (*ST*) Estimated basin lagtime (*LT*) and peak discharge (Q_p) are applied to a dimensionless hydrograph to estimate a typical flood hydrograph for the given peak discharge If the peak discharge has to be estimated, equations 1 through 6 (table 8) could be applied

Because the dimensionless hydrograph was developed from events of approximately average duration, the procedure outlined above will generate a simulated hydrograph of approximately average duration. The reader is cautioned that actual floods of similar peak discharge but considerably longer duration (and greater volume) also are possible

Limitations of the Method

The method is limited to ungaged sites that have basin characteristics similar to those of the 30 gaged sites used in the peak and lagtime regression analyses and dimensionless hydrograph verification

The ranges of the explanatory variables in the peak and lagtime regression analyses are listed in the following table.

Variable	Minimum	Maximum	Unit
A	0 026	4 09	square miles
P	31 5	41 2	inches
BDF	0	12	scale from 0 to 12
SL	8 00	462	feet per mile
L	300	4 50	miles

Application of the method to streams having basin characteristics outside of these ranges may result in errors that are considerably greater than those implied by the error analyses

Additional limitations of the hydrograph estimation technique include the limitations described in the section "Application of Peak-Frequency Equations "

Computation of Basin Characteristics

The values of the basin characteristics of the ungaged site are entered into the appropriate regression equations to compute peak discharge for the desired recurrence interval and basin lagtime Values for *A*, *P*, and *BDF* may be determined as described in the section "Application of Peak-Frequency Equations." Values for *SL* and *L* are determined as follows

SL Main-channel slope (in feet per mile)—Computed as the difference in elevation (in feet) at points 10 and 85 percent of the distance along the main channel from a specified location on the channel to the topographic divide for the contributing drainage area, divided by the channel distance (in miles) between the two points, as determined from USGS 7 5-minute topographic quadrangle maps or sewer maps (fig 7)

L Main-channel length (in miles)—Computed as the distance measured along the main channel from the ungaged site to the basin divide for the contributing drainage area, as determined from USGS 7 5-minute topographic quadrangle maps or sewer maps (fig 7)

Computation of Peak Discharge

The following procedure may be used if it is necessary to estimate the peak discharge for hydrograph estimation

1 Determine the values of *A*, *P*, and *BDF* as described in the section "Application of Peak-Frequency Equations "
2 Check that the characteristics of the basin meet the criteria described in "Limitations of the Method" in the section "Application of Peak-Frequency Equations "
3 Select the appropriate equation from table 8 for the desired recurrence interval
4 Substitute the computed values of *A*, *P*, and *BDF* into the equation
5 Compute the peak discharge.

Computation of Basin Lagtime

The following procedure should be used for estimating the basin lagtime of small urban streams in Ohio

1 Determine the values of *SL*, *L*, and *BDF*, as described above
2 Check that the characteristics of the basin meet the criteria described above
3 Substitute the values of *SL*, *L*, and *BDF* into equation 7
4 Compute the basin lagtime.

Computation and Plotting of Flood Hydrograph

The following procedure may be used to estimate flood hydrographs having a specific peak discharge for small urban streams in Ohio

1 If it is necessary to estimate the peak discharge (Q_p), use the procedure described above
2 Estimate the basin lagtime (*LT*) by use of the procedure described above
3 Multiply each value of *t/LT* in table 10 by *LT* These computed values are the time (*t*) coordinates of the hydrograph $t = (t/LT)(LT)$
4 Multiply each value of Q/Q_p in table 10 by Q_p These computed values are the corresponding discharge (*Q*) coordinates of the hydrograph
$Q = (Q/Q_p)(Q_p)$
5 Plot time (*t*) against discharge (*Q*)

Example of Computation of Flood Hydrograph

Estimate the flood hydrograph of the 100-year flood for an ungaged urban stream in Toledo, Ohio (fig 7), where

A = 0 89 square miles
P = 31 6 inches
BDF = 9
SL = 16 3 feet per mile, and
L = 1 36 miles

These values are within the ranges of the explanatory variables used in the development of the hydrograph-estimation method

1 The 100-year flood peak discharge is estimated by use of equation 6 (table 8)
$UQ_{100} = 321\ (A)^{0\ 79}(P{-}30)^{0\ 76}(13{-}BDF)^{-0\ 33}$
$UQ_{100} = 321\ (0\ 89)^{0\ 79}(31\ 6{-}30)^{0\ 76}(13{-}9)^{-0\ 33}$
$UQ_{100} = Q_p = 265$ cubic feet per second
2 The basin lagtime is estimated by use of equation 7·
$LT = 1\ 13\ (L/\sqrt{SL})^{0\ 57}(13{-}BDF)^{0\ 46}$
$LT = 1\ 13\ (1\ 36/\sqrt{16\ 3})^{0\ 57}(13{-}9)^{0\ 46}$
$LT = 1\ 15$ hours

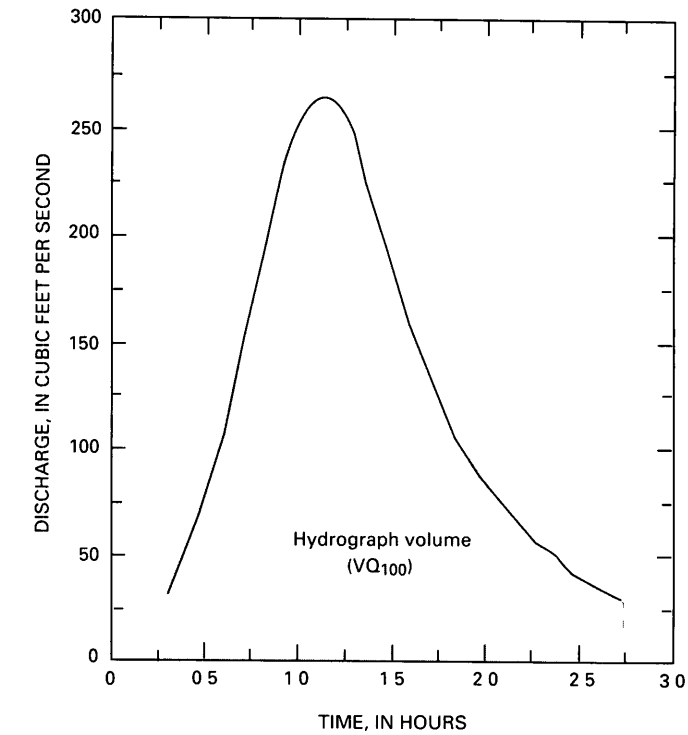


Figure 14. Estimated flood hydrograph for 100-year peak discharge for an ungaged urban stream in Toledo, Ohio

3 Each value of *t/LT* in table 10 is multiplied by 1 15 hours (Results are presented in table 11)
4 Each value of Q/Q_p in table 9 is multiplied by 265 cubic feet per second (Results are presented in table 11)
5 Time (*t*) versus discharge (*Q*) is plotted (fig 14)

Computation of Hydrograph Volume

Flood volume corresponding to the estimated hydrograph may be computed by numerically integrating the area under the hydrograph or by use of an equation developed in this section The two methods yield identical results The computed volume is an average or typical volume for the design peak discharge

The cumulative volume (*VQ*) indicated in table 11 is computed by multiplying the time-ratio increment (0 05) times the lagtime (1 15 hours) times 3,600 seconds per hour times the mean discharge (*Q*) for the time increment as shown in the following example for the first increment

$$VQ = (\ 05)\ (1\ 15)\ (3{,}600)\ [(31\ 8 + 42\ 4)/2]$$
$$VQ = 7{,}680 \text{ cubic feet}$$

These values are summed to compute the total volume (VQ_{100}) of 1,143,000 cubic feet The total

Table 11. Computation of estimated hydrograph and integration of flood volume of estimated100-year peak discharge for an ungaged urban stream in Toledo, Ohio

[ft^3, cubic feet, ft^3/sec, cubic feet per second]

t/LT ×	*LT* =	*t*	*Q/Qp* ×	*Qp* =	*Q*	*VQ*
Time ratio	From step 2	Time, hours	Discharge ratio	From step 1	Discharge, ft^3/sec	Cumulative volume, ft^3
0 25	1 15	0 29	0 12	265	31 8	0
30	1 15	35	16	265	42 4	7,680
35	1 15	40	21	265	55 7	17,800
40	1 15	46	26	265	68 9	30,700
45	1 15	52	33	265	87 5	46,900
50	1 15	58	40	265	106	66,900
55	1 15	63	49	265	130	91,400
60	1 15	69	58	265	154	121,000
65	1 15	75	67	265	178	155,000
70	1 15	81	76	265	201	194,000
75	1 15	86	84	265	223	238,000
80	1 15	92	90	265	239	286,000
85	1 15	98	95	265	252	337,000
90	1 15	1 04	98	265	260	390,000
95	1 15	1 09	1 00	265	265	444,000
1 00	1 15	1 15	99	265	262	499,000
1 05	1 15	1 21	96	265	254	552,000
1 10	1 15	1 27	92	265	244	604,000
1 15	1 15	1 32	86	265	228	653,000
1 20	1 15	1 38	80	265	212	698,000
1 25	1 15	1 44	74	265	196	740,000
1 30	1 15	1 50	68	265	180	779,000
1 35	1 15	1 55	62	265	164	815,000
1 40	1 15	1 61	56	265	148	847,000
1 45	1 15	1 67	51	265	135	876,000
1 50	1 15	1 73	47	265	125	903,000
1 55	1 15	1 78	43	265	114	928,000
1 60	1 15	1 84	39	265	103	951,000
1 65	1 15	1 90	36	265	95 4	971,000
1 70	1 15	1 96	33	265	87 5	990,000
1 75	1 15	2 01	30	265	79 5	1,007,000
1 80	1 15	2 07	28	265	74 2	1,023,000
1 85	1 15	2 13	26	265	68 9	1,038,000
1 90	1 15	2 19	24	265	63 6	1,052,000
1 95	1 15	2 24	22	265	58 3	1,064,000
2 00	1 15	2 30	20	265	53 0	1,076,000
2 05	1 15	2 36	19	265	50 4	1,087,000
2 10	1 15	2 42	17	265	45 1	1,096,000
2 15	1 15	2 47	16	265	42 4	1,105,000
2 20	1 15	2 53	15	265	39 8	1,114,000
2 25	1 15	2 59	14	265	37 1	1,122,000
2 30	1 15	2 65	13	265	34 5	1,129,000
2 35	1 15	2 70	12	265	31 8	1,136,000
2 40	1 15	2 76	11	265	29 2	1,143,000
	Duration (D)=2 47 hours				Total volume (VQ_{100})=1,143,000 ft^3	

volume, which is indicated by the shaded area in figure 14, does not include the volume under the tails of the hydrograph To quickly compute the total volume (VQ_p), use the following equation

$$VQ_p = 3{,}750\ (Q_p)\ (LT), \qquad (8)$$

where

VQ_p is hydrograph volume of Q_p (in cubic feet),

Q_p is peak discharge (in cubic feet per second), and

LT is basin lagtime (in hours)

The constant (3,750) in equation 8 is the difference between the last and first time ratios (2 40 – 0 25 = 2 15) times 3,600 seconds per hour times the mean of the incremental discharge ratios (0 484)

$$(2\ 15)\ (3{,}600)\ (0\ 484)\ =\ 3{,}750$$

Example

$VQ_{100} = 3{,}750(Q_{100})(LT)$

$VQ_{100} = 3{,}750(265)(1\ 15)$

$VQ_{100} = 1{,}143{,}000$ cubic feet

The duration (D) of the simulated hydrograph may be computed by use of the following equation

$$D = 2\ 15\ (LT), \qquad (9)$$

where

D is hydrograph duration (in hours), and

LT is basin lagtime (in hours)

The constant (2 15) in equation 9 is the difference between the last and first time ratios

$$(2\ 40 - 0\ 25 = 2\ 15)$$

Example

$D = 2\ 15\ (LT)$

$D = 2\ 15\ (1\ 15)$

$D = 2\ 47$ hours

ESTIMATION OF VOLUME-DURATION-FREQUENCY RELATIONS AT UNGAGED URBAN SITES

Previous sections of this report describe the development and application of methods for estimating flood peak discharges and corresponding flood hydrographs Such methods may provide the necessary inflow information for the design of hydraulic structures for which temporary storage of water upstream from the structure is not considered to be an important factor This section of the report describes a method applicable to situations where the design-peak outflow is required or desired to be less than the design-peak inflow. In this case, some volume of water must temporarily be stored upstream from the structure, and an estimate of the maximum volume for a design duration and recurrence interval is needed

Development of Volume-Duration-Frequency Equations

Multiple-regression techniques similar to those used in development of the peak-frequency equations were used to develop equations for estimating volume-duration-frequency relations of small urban streams in Ohio The volume-duration-frequency data for the 62 urban and rural study sites (table 7) were used in the analysis The reasons for combining the urban and rural data into a single data set for the volume analyses were previously discussed The analysis resulted in 36 equations where flood volumes of specific duration and recurrence interval are the response variables and drainage area (A), average annual precipitation (P), and basin-development factor (BDF) are the explanatory variables

Flood Volumes as a Function of Basin Characteristics

Flood-volume data for all combinations of the six durations (1, 2, 4, 8, 16, and 32 hours) and six recurrence intervals (2, 5, 10, 25, 50, and 100 years) were analyzed as a function of basin characteristics The volume-duration-frequency data can be identified by abbreviations in the form dV_T, in which V is total volume, in millions of cubic feet, d is duration, in hours, and T is recurrence interval, in years For example, $4V_{50}$ identifies the maximum 4-hour volume with a 50-year recurrence interval The 36 volume-duration-frequency data sets (response variables) were initially related to a variety of basin characteristics (explanatory variables) in the multiple-regression analysis

The basin characteristics tested were

A —drainage area

BDF —basin-development factor

IA —impervious area

L —main-channel length

SL —main-channel slope

$L/\sqrt{SL}$ —main-channel length divided by the square root of the main-channel slope
F —forested area
P —average annual precipitation
ST —storage area
$2RF_{25}$ —2-hour, 25-year rainfall
$2RF_{100}$ —2-hour, 100-year rainfall
$6RF_{25}$ —6-hour, 25-year rainfall
$6RF_{100}$ —6-hour, 100-year rainfall
$12RF_{25}$ —12-hour, 25-year rainfall
$12RF_{100}$ —12-hour, 100-year rainfall

The analysis yielded the 36 regression equations listed in table 12 The equations can be used to estimate maximum volumes of specific recurrence interval and duration for small urban streams in Ohio All equations are subject to limitations discussed in subsequent parts of this report Also listed in table 12 are the average standard error of regression (SER) and average standard error of prediction (SEP) for each equation

The same explanatory variables (*A*, *P*, and *BDF*) that are statistically significant in the peak-frequency equations (table 8) are also statistically significant in the volume-duration-frequency equations for the 1-, 2-, and 4-hour durations (table 12). *A* and *P* were significant for the 8-, 16-, and 32-hour durations The values of *A*, *P*, and *BDF* for the 62 study sites are listed in table 13. The same transformations that were applied for the peak-frequency analysis were also applied for the volume-duration-frequency analysis *BDF* was significant only for the 1-, 2-, and 4-hour durations Basin development generally affects the magnitude of the peak discharge and shape of the runoff hydrograph more than it affects the total runoff volume It therefore seems reasonable that *BDF* would not be statistically significant in equations for the long durations which estimate a larger part of the total runoff hydrograph than do equations for short durations All three explanatory variables (*A*, *P*, and *BDF*) had median significance levels equal to or less than 1 percent

A minimum attained significance level of 10 percent was met for all 36 equations for *A* and *P*, and for all 1- and 2-hour equations for *BDF* For the six 4-hour equations, attained significance levels for *BDF* however, ranged from 8 to 45 percent *BDF* was, however, included in the 4-hour equations to provide a smooth transition from the 2-hour volume to the 8-hour volume when plotting an estimated volume-duration-frequency curve

Sensitivity Analysis

Errors in measurement or judgment may occur when determining values for the explanatory variables (*A*, *P*, and *BDF*) Consequently, a sensitivity analysis was performed to illustrate the effects of errors in the explanatory variables on the computations of flood volumes (refer to page 16 for a general discussion of sensitivity analyses) The means of the three explanatory variables for the 30 urban study sites were calculated to be

$A = 0\ 778$ square miles,
$P = 37\ 3$ inches,
$BDF = 7\ 97$

These values were substituted into the 36 regression equations Each explanatory variable was then varied by 5-percent increments from −50 percent to +50 percent of its mean while the values of the other variables were held constant The percentage of change in the explanatory variable was then plotted against the percentage of change in the computed volumes The results are presented in figure 15 (Because all 36 plots were similar, only 9 representative plots are shown)

The plots indicate that the sensitivity of computed volume to changes in drainage area (*A*) increases with an increase in duration. The sensitivity of computed volume to changes in average annual precipitation (*P*) decreases slightly with an increase in duration The sensitivity of computed volume to changes in *A* and *P* is fairly constant with respect to recurrence interval As was evident in the sensitivity analysis for the peak-frequency equations, the sensitivity of computed volume to changes in basin-development factor (*BDF*) decreases slightly with an increase in recurrence interval, and increases with positive changes in *BDF* The sensitivity of computed volume to changes in *BDF* decreases significantly as duration increases from 1 hour to 4 hours

Tests for Intercorrelation and Bias

The same tests for intercorrelation, parametrical bias, and geographical bias that were performed for the peak-frequency equations also were performed for the volume-duration-frequency equations These tests indicated that the 36 volume-duration-frequency equations are not appreciably affected by intercorrelation of explanatory variables, parametrical bias, or geographical bias

Table 12 Equations for estimating volume-duration-frequency (dV_T) relations of small urban streams in Ohio

[SER, average standard error of regression (in percent), SEP, average standard error of prediction (in percent), dV_T, flood volume of d hours duration and T years recurrence interval (in millions of cubic feet), A, drainage area (in square miles), P, average annual precipitation (in inches), BDF, basin-development factor (on a scale from 0 to 12)]

Equation number	Equation	SER (in percent)	SEP (in percent)
	2-year equations		
(10)	$1V_2 = 0.42(A)^{0.77}(P\text{-}30)^{0.43}(13\text{-}BDF)^{-0.41}$	±38.1	±39.4
(11)	$2V_2 = 0.57(A)^{0.81}(P\text{-}30)^{0.38}(13\text{-}BDF)^{-0.25}$	±37.0	±38.4
(12)	$4V_2 = 0.70(A)^{0.85}(P\text{-}30)^{0.33}(13\text{-}BDF)^{-0.11}$	±36.3	±37.9
(13)	$8V_2 = 0.79(A)^{0.89}(P\text{-}30)^{0.32}$	±37.3	±39.0
(14)	$16V_2 = 0.96(A)^{0.93}(P\text{-}30)^{0.32}$	±39.6	±41.4
(15)	$32V_2 = 1.11(A)^{0.95}(P\text{-}30)^{0.32}$	±41.7	±43.7
	5-year equations		
(16)	$1V_5 = 0.60(A)^{0.76}(P\text{-}30)^{0.49}(13\text{-}BDF)^{-0.38}$	±35.1	±36.4
(17)	$2V_5 = 0.80(A)^{0.80}(P\text{-}30)^{0.42}(13\text{-}BDF)^{-0.22}$	±32.9	±34.2
(18)	$4V_5 = 0.97(A)^{0.84}(P\text{-}30)^{0.39}(13\text{-}BDF)^{-0.06}$	±31.1	±32.6
(19)	$8V_5 = 1.19(A)^{0.90}(P\text{-}30)^{0.37}$	±31.5	±33.2
(20)	$16V_5 = 1.45(A)^{0.94}(P\text{-}30)^{0.37}$	±34.2	±36.0
(21)	$32V_5 = 1.63(A)^{0.95}(P\text{-}30)^{0.39}$	±36.8	±38.7
	10-year equations		
(22)	$1V_{10} = 0.74(A)^{0.76}(P\text{-}30)^{0.51}(13\text{-}BDF)^{-0.37}$	±34.8	±36.2
(23)	$2V_{10} = 0.98(A)^{0.80}(P\text{-}30)^{0.45}(13\text{-}BDF)^{-0.20}$	±32.0	±33.4
(24)	$4V_{10} = 1.19(A)^{0.84}(P\text{-}30)^{0.40}(13\text{-}BDF)^{-0.05}$	±29.6	±31.2
(25)	$8V_{10} = 1.52(A)^{0.90}(P\text{-}30)^{0.38}$	±29.4	±31.0
(26)	$16V_{10} = 1.85(A)^{0.94}(P\text{-}30)^{0.38}$	±32.1	±33.9
(27)	$32V_{10} = 2.05(A)^{0.96}(P\text{-}30)^{0.41}$	±34.8	±36.7
	25-year equations		
(28)	$1V_{25} = 0.94(A)^{0.76}(P\text{-}30)^{0.52}(13\text{-}BDF)^{-0.37}$	±35.1	±36.6
(29)	$2V_{25} = 1.24(A)^{0.80}(P\text{-}30)^{0.46}(13\text{-}BDF)^{-0.19}$	±31.7	±33.2
(30)	$4V_{25} = 1.51(A)^{0.84}(P\text{-}30)^{0.41}(13\text{-}BDF)^{-0.04}$	±28.7	±30.3
(31)	$8V_{25} = 2.01(A)^{0.90}(P\text{-}30)^{0.38}$	±27.5	±29.2
(32)	$16V_{25} = 2.48(A)^{0.95}(P\text{-}30)^{0.37}$	±30.0	±31.8
(33)	$32V_{25} = 2.66(A)^{0.96}(P\text{-}30)^{0.42}$	±33.0	±35.0

Application of Volume-Duration-Frequency Equations

The volume-duration-frequency equations for the desired recurrence interval can be applied to develop a relation between inflow volume and duration for an ungaged site. A theoretical maximum-volume hydrograph based on the volume-duration data can be constructed by converting the volume data as a function of duration to discharge data as a function of time and plotting the discharge data in a symmetrical pattern centered about the peak. This hydrograph can be used to develop a relation between inflow volume and time. This relation, in combination with an estimate of the relation between outflow volume and time for a hydraulic structure, can be used to develop an

Table 12 Equations for estimating volume-duration-frequency (dV_T) relations of small urban streams in Ohio—Continued

Equation number	Equation				SER (in percent)	SEP (in percent)
			50-year equations			
(34)	$1V_{50}$	$= 1.10(A)^{0.76}$	$(P\text{-}30)^{0.52}$	$(13\text{-}BDF)^{-0.36}$	±35.6	±37.2
(35)	$2V_{50}$	$= 1.46(A)^{0.80}$	$(P\text{-}30)^{0.46}$	$(13\text{-}BDF)^{-0.19}$	±31.9	±33.4
(36)	$4V_{50}$	$= 1.79(A)^{0.84}$	$(P\text{-}30)^{0.41}$	$(13\text{-}BDF)^{-0.04}$	±28.4	±30.1
(37)	$8V_{50}$	$= 2.43(A)^{0.90}$	$(P\text{-}30)^{0.38}$		±26.5	±28.2
(38)	$16V_{50}$	$= 3.04(A)^{0.95}$	$(P\text{-}30)^{0.36}$		±28.9	±30.7
(39)	$32V_{50}$	$= 3.18(A)^{0.96}$	$(P\text{-}30)^{0.43}$		±32.2	±34.1
			100-year equations			
(40)	$1V_{100}$	$= 1.28(A)^{0.77}$	$(P\text{-}30)^{0.51}$	$(13\text{-}BDF)^{-0.36}$	±36.2	±37.9
(41)	$2V_{100}$	$= 1.69(A)^{0.80}$	$(P\text{-}30)^{0.45}$	$(13\text{-}BDF)^{-0.19}$	±32.1	±33.8
(42)	$4V_{100}$	$= 2.10(A)^{0.84}$	$(P\text{-}30)^{0.41}$	$(13\text{-}BDF)^{-0.04}$	±28.4	±30.2
(43)	$8V_{100}$	$= 2.92(A)^{0.91}$	$(P\text{-}30)^{0.36}$		±25.8	±27.5
(44)	$16V_{100}$	$= 3.61(A)^{0.95}$	$(P\text{-}30)^{0.36}$		±27.8	±29.6
(45)	$32V_{100}$	$= 3.77(A)^{0.96}$	$(P\text{-}30)^{0.42}$		±31.4	±33.3

estimate of the relation between required storage and time

Limitations of the Method

The 36 multiple-regression equations developed for estimating volume-duration-frequency relations are applicable to sites on small urban streams in Ohio whose basin characteristics are within the ranges of the basin characteristics of the sites used in the regression analysis The following table shows the ranges of the basin characteristics of the 62 study sites.

Basin characteristic	Minimum	Maximum	Unit
A	0.026	6.45	square miles
P	31.5	42.8	inches
BDF	0	12	scale from 0 to 12

Application of the equations to streams having basin characteristics outside of these ranges may result in errors that are considerably greater than those implied by the standard error of prediction

All study sites were chosen to have minimal basin storage (mean storage area for the 62 study sites was 0.26 percent of total drainage area, the maximum value was 3.1 percent). The equations may not be applicable to streams whose flood characteristics are significantly affected by storage or where upstream culverts or other structures might attenuate the peak discharges Storage upstream of the ungaged site will generally affect short-duration volumes more than long-duration volumes

It was assumed that annual-peak volumes (for all durations) of small streams in Ohio are caused by rain falling on unfrozen ground, usually during 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 volumes are likely to be affected by snowmelt or frozen ground

Computation of Basin Characteristics

The values of the basin characteristics of the ungaged site are entered into the appropriate equations to compute the volume-duration relations for the desired recurrence interval Values for A, P, and BDF can be determined as described previously

Computation of Flood Volumes as a Function of Duration

The following steps describe the procedure used to estimate volume-duration-frequency (dV_T) relations of small urban streams in Ohio

Table 13. Values of the significant explanatory variables in the volume-duration-frequency equations for 62 study sites in Ohio

[*A*, drainage area (in square miles), *P*, average annual precipitation (in inches), *BDF*, basin-development factor (on a scale from 0 to 12)]

Station name	*A*	P	BDF	Station name	*A*	P	BDF
Amberly Ditch	0 14	39 8	9	Ketchum Ditch	0 84	31 5	10
Anderson Ditch	049	40 1	8	King Run	53	35 4	0
Barnes Run	1 02	40 1	0	Kitty Creek	1 75	37 2	0
Browns Run	2 00	35 4	0	Mall Run	16	38 5	12
Bull Creek	3 13	39 0	0	March Run	18	36 8	0
Bunn Brook	51	35 6	8	Norman Ditch	60	37 2	10
Carter Creek	1 16	34 7	0	North Fork Doan Brook	1 18	39 1	10
Cattail Creek	13	36 9	0	Orchard Run	43	36 9	11
Charles Ditch	50	35 3	11	Pike Run	1 18	35 8	7
Chestnut Creek	22	41 3	0	Racetrack Run	34	34 0	0
Claypit Creek	2 25	39 1	0	Rand Run	33	38 3	4
Coalton Ditch	50	41 2	0	Reitz Run	98	31 9	0
Dawnlight Ditch	20	36 8	8	Rush Run	72	36 6	2
Delhi Ditch	16	40 1	10	Sandhill Creek	1 76	35 6	0
Delwood Run	45	35 0	0	Sandusky Creek	73	41 8	0
Dugway Brook	1 42	39 0	12	Second Creek	1 04	38 8	0
Duncan Hollow Creek	51	41 6	0	Silver Creek	4 09	31 6	6
Dundee Creek	74	37 5	0	Slim Creek	13	38 4	0
Elk Fork	6 45	42 5	0	Springfield Ditch	26	39 8	9
Elk Run	48	40 7	0	Stone Branch	84	42 1	0
Euclid Creek Tributary	1 67	39 4	11	Stripe Creek	1 26	36 0	0
Falling Branch	33	38 3	0	Sugar Run	1 37	42 8	0
Fire Run	24	40 9	0	Sweet Henri Ditch	36	36 7	5
Fishinger Creek	66	37 2	9	Tifft Ditch	85	31 7	8
Fishinger Road Creek	45	37 1	11	Tinkers Creek Tributary	12	40 5	3
Gentile Ditch	064	39 2	12	Tombstone Creek	4 03	36 9	0
Glen Park Creek	1 21	33 8	4	Trippetts Branch	33	38 2	0
Grassy Creek	1 81	31 7	6	Twist Run	65	40 0	0
Harte Run	86	37 0	0	Whipps Ditch	2 64	40 3	9
Home Ditch	24	39 9	3	Wolfkiln Run	87	40 3	0
Hoskins Creek	5 42	42 5	0	Wyoming Ditch	026	39 7	11

1 Determine the values of *A*, *P*, and *BDF*, as described previously in "Computation of Basin Characteristics" in the section "Application of Peak-Frequency Equations "
2 Check that the characteristics of the basin meet the criteria described above in "Limitations of the Method "
3 Select the appropriate equations from table 12 for the desired recurrence interval
4 Substitute the values of *A*, *P*, and *BDF* into the equations
5 Compute the flood volumes
6 Plot the flood volumes as a function of duration

Example of Computation of Flood Volume

Estimate the 100-year flood volumes for all six durations for an ungaged urban stream in Toledo, Ohio (fig 7)

1 The following basin characteristics are determined

$A =$ 0 89 square miles

$P =$ 31 6 inches

$BDF = 9$

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

3 The appropriate equations to be applied from table12 are

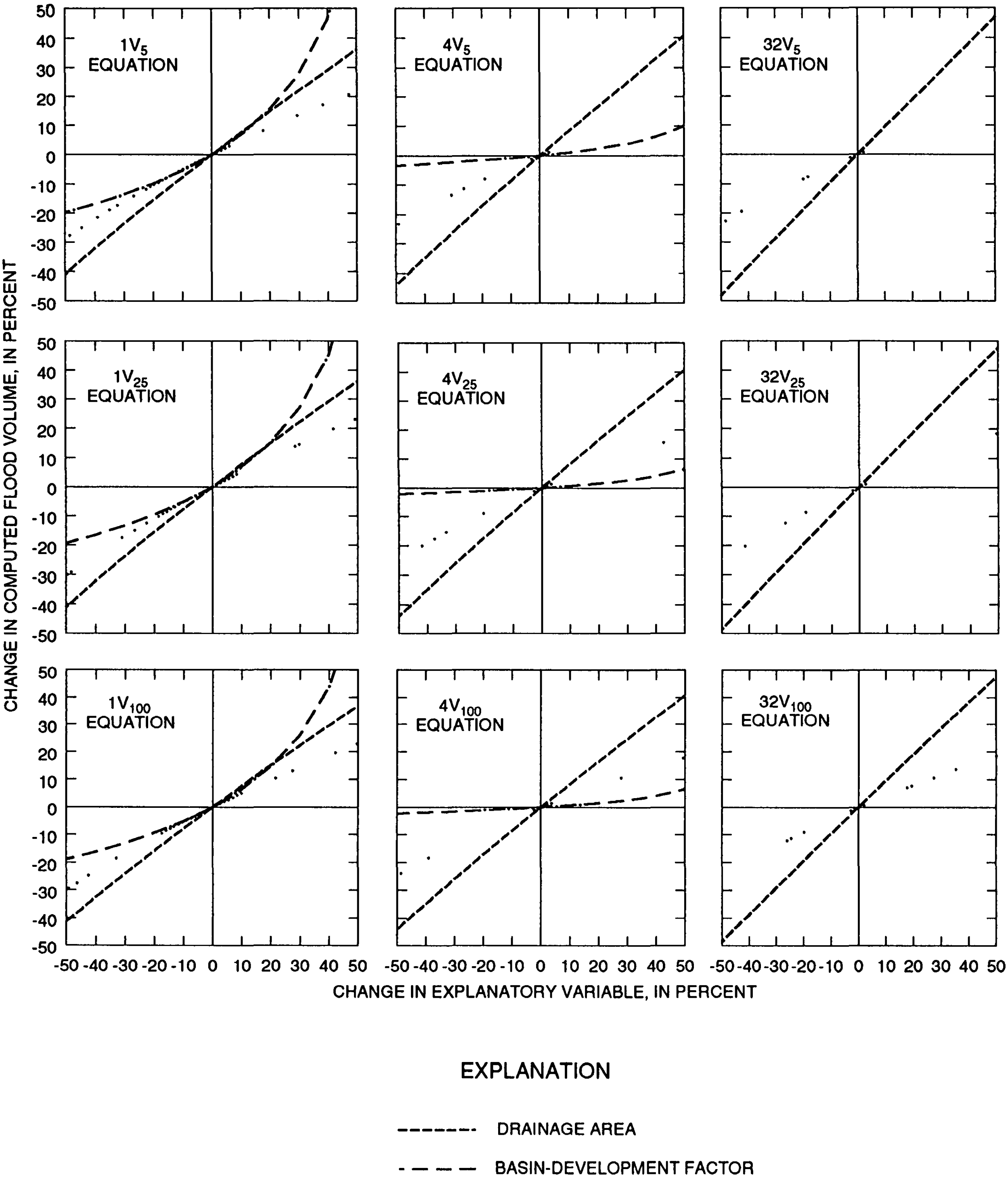


Figure 15. Sensitivity of computed flood volumes to changes from the means of the explanatory variables in the volume-duration-frequency (dV_T) equations for selected durations and recurrence intervals

$1V_{100}=1.28(A)^{0.77}(P-30)^{0.51}(13-BDF)^{-0.36}$
$2V_{100}=1.69(A)^{0.80}(P-30)^{0.45}(13-BDF)^{-0.19}$
$4V_{100}=2.10(A)^{0.84}(P-30)^{0.41}(13-BDF)^{-0.04}$
$8V_{100}=2.92(A)^{0.91}(P-30)^{0.36}$
$16V_{100}=3.61(A)^{0.95}(P-30)^{0.36}$
$32V_{100}=3.77(A)^{0.96}(P-30)^{0.42}$

4. The basin characteristics are substituted into the equations

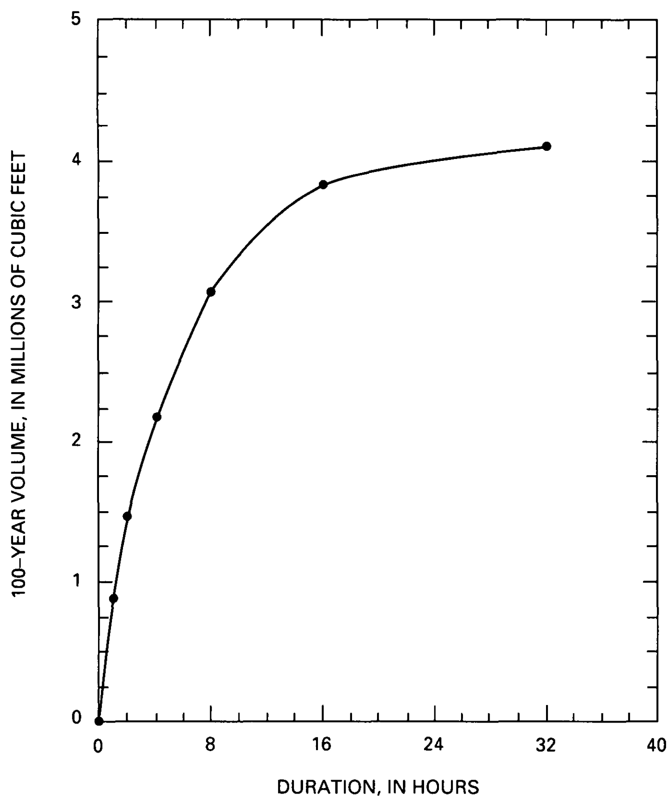


Figure 16 Estimated 100-year volumes as a function of duration for an ungaged urban stream in Toledo, Ohio

$1V_{100}=1\ 28(0\ 89)^{0\ 77}(31\ 6-30)^{0\ 51}(13-9)^{-0\ 36}$
$2V_{100}=1\ 69(0\ 89)^{0\ 80}(31\ 6-30)^{0\ 45}(13-9)^{-0\ 19}$
$4V_{100}=2\ 10(0\ 89)^{0\ 84}(31\ 6-30)^{0\ 41}(13-9)^{-0\ 04}$
$8V_{100}=2\ 92(0\ 89)^{0\ 91}(31\ 6-30)^{0\ 36}$
$16V_{100}=3\ 61(0\ 89)^{0\ 95}(31\ 6-30)^{0\ 36}$
$32V_{100}=3\ 77(0\ 89)^{0\ 96}(31\ 6-30)^{0\ 42}$

5 The estimated 100-year flood volumes are
$1V_{100}=0\ 90$ million cubic feet
$2V_{100}=1.46$ million cubic feet
$4V_{100}=2\ 18$ million cubic feet
$8V_{100}=3\ 11$ million cubic feet
$16V_{100}=3\ 83$ million cubic feet
$32V_{100}=4\ 11$ million cubic feet

6 The estimated volumes can then be plotted as a function of duration to yield a curve showing inflow volume as a function of duration as shown in figure 16 The lines connecting the symbols in figure 16 are for illustration purposes only

Computation of Flood Volumes as a Function of Time

Depending on the design application, it may be desirable to convert the volume data as a function of duration (dV_T) to cumulative volume data as a function of time ($VQ_T(t)$) for a hypothetical hydrograph having the same volume-duration characteristics A method is illustrated in figure 17 that is based on the assumption of a hypothetical maximum-volume hydrograph, which can be derived from the volume-duration data and constructed by converting the volume data as a function of duration to discharge data as a function of time and plotting the discharge data in a symmetrical pattern centered about the peak Computation of the $VQ_T(t)$ data from the dV_T data is shown in table 14 The method, if applied, should be based on the entire 32-hour volume-duration-frequency curve The cumulative volume data as a function of time ($VQ_T(t)$) may then be plotted as shown in figure 18 The hydrograph in figure 17 is analogous to the hydrograph in figure 4, which illustrates the selection of volumes for each of the six durations However, to simplify the computations, the hydrograph in figure 17 has been constructed symmetrically and in a bar graph form. An actual hydrograph of such long duration would probably be asymmetrical The figure 17 hydrograph is also based on the assumption that the maximum volumes for all six durations came from the same flood event In fact, this is often, but not always, true Thus, the cumulative volume data plotted in figure 18 is an approximation based on these assumptions

COMPARISON OF VOLUME-ESTIMATION TECHNIQUES

The preceding sections describe two methods for estimating flood volumes Figure 19 is a graph showing the volume-duration curve estimated from the 100-year volume-duration-frequency (dV_{100}) equations (eq 40–45) and the volume-duration curve estimated by integrating under the estimated hydrograph for the 100-year peak discharge (VQ_{100}, fig 14) for an ungaged urban stream in Toledo, Ohio Both curves represent the estimated maximum volume for the indicated duration as illustrated in figure 4 In the example shown in figure 19, both methods of volume computation produce similar results up to a duration of about 2 hours The VQ_{100} curve ends at 2 47 hours (total duration (D) of the simulated hydrograph) with a relatively small increase in volume from 2 to 2.47 hours The dV_{100} curve ends at 32 hours with a significant increase in volume from 2 to 16 hours

Estimates of volume obtained by application of the volume-duration-frequency (dV_T) equations are not intended to replace the volume estimates obtained

Table 14. Computations of cumulative volume as a function of time ($VQ_T(t)$) from volume as a function of duration (dV_T) for an ungaged urban stream in Toledo, Ohio

[d, duration (in hours), t, time (in hours), dV_T, volume (in millions of cubic feet) of d hours duration and T years recurrence interval, $VQ_T(t)$, cumulative volume (in millions of cubic feet) of t hours time and T years recurrence interval]

d or t	**dV_T**	**$VQ_T(t)$ Equation**				**Computation**		**$VQ_T(t)$**
0	0 0	$VQ_{100}(0)$	=	$0V_{100}$	=	0	=	0
1	0 90	—		—		—		—
2	1 46	—		—		—		—
4	2 18	—		—		—		—
8	3 11	$VQ_{100}(8)$	=	$\frac{1}{2}(32V_{100}-16V_{100})$	=	$\frac{1}{2}$ (4 11 − 3 83)	=	0 14
12	—	$VQ_{100}(12)$	=	$\frac{1}{2}(32V_{100}-8V_{100})$	=	$\frac{1}{2}$ (4 11 − 3 11)	=	0 50
14	—	$VQ_{100}(14)$	=	$\frac{1}{2}(32V_{100}-4V_{100})$	=	$\frac{1}{2}$ (4 11− 2 18)	=	0 96
15	—	$VQ_{100}(15)$	=	$\frac{1}{2}(32V_{100}-2V_{100})$	=	$\frac{1}{2}$ (4 11 − 1 46)	=	1 32
16	3 83	$VQ_{100}(16)$	=	$\frac{1}{2}(32V_{100})$	=	$\frac{1}{2}$ (4 11)	=	2 06
17	—	$VQ_{100}(17)$	=	$32V_{100}-VQ_{100}(15)$	=	4 11 − 1 32	=	2 79
18	—	$VQ_{100}(18)$	=	$32V_{100}-VQ_{100}(14)$	=	4 11 − 0 96	=	3 15
20	—	$VQ_{100}(20)$	=	$32V_{100}-VQ_{100}(12)$	=	4 11 − 0 50	=	3 61
24	—	$VQ_{100}(24)$	=	$32V_{100}-VQ_{100}(8)$	=	4 11 − 0 14	=	3 97
32	4 11	$VQ_{100}(32)$	=	$32V_{100}$	=	4 11	=	4 11

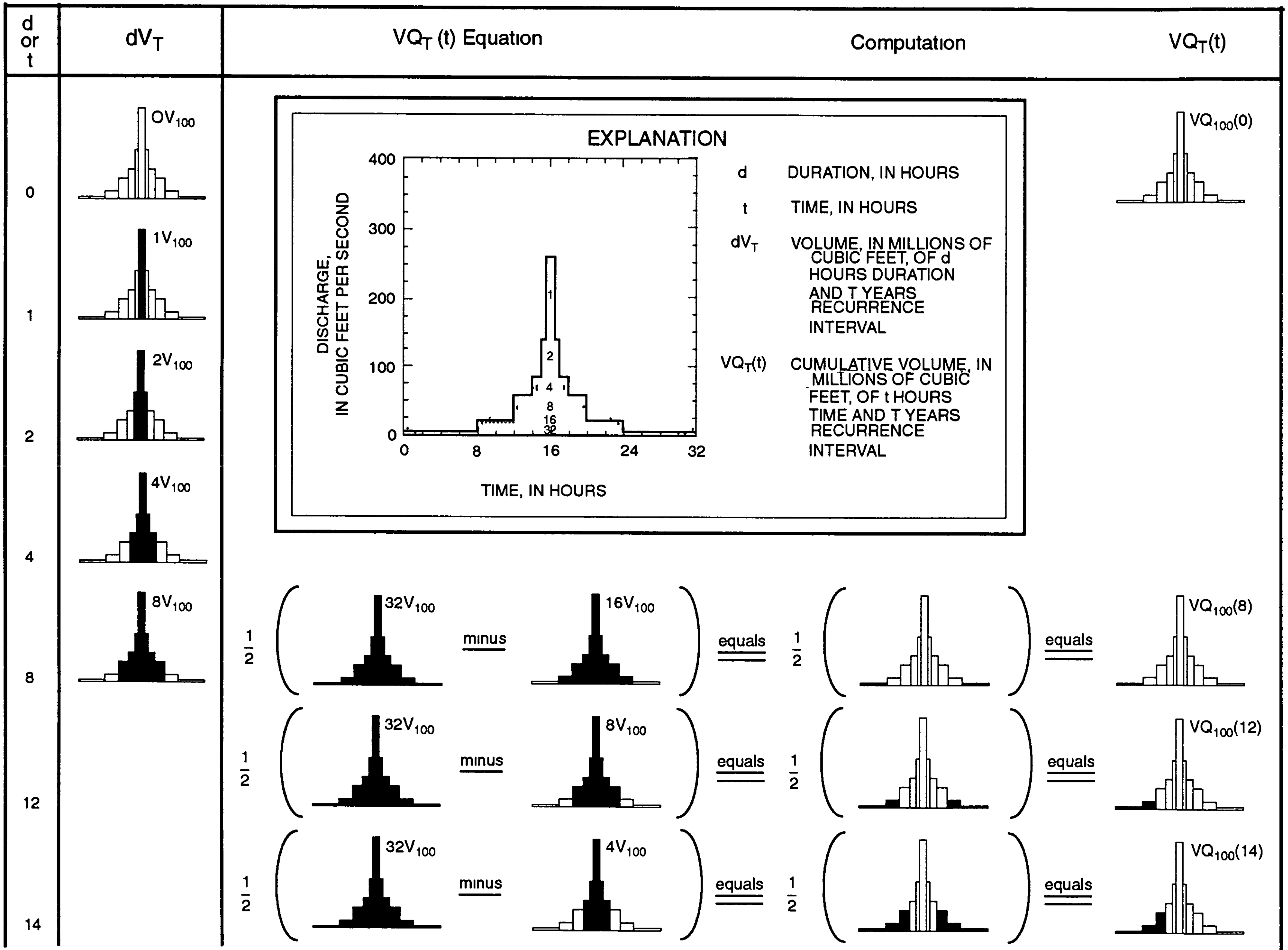
by integrating the area under an estimated hydrograph, but rather to provide additional information for design situations in which inflow and outflow rates for a hydraulic structure may not be equal Both methods yield similar results for volume estimates of short duration The dV_T equations can be used to compute volume estimates of long and short duration because the dV_T equations are based on maximum-annual-volume data of long and short duration The dimensionless-hydrograph method is based on flood hydrographs of average duration and cannot be used to compute volume estimates of long duration Volume estimates of long duration may be considerably greater than volume estimates of short duration It may be necessary to estimate flood hydrographs for many design situations because the hydrographs provide a means of routing discharges through a hydraulic structure, so that concurrent outflow discharges can be estimated

The two methods, in effect, provide estimates of resultant runoff volumes from two different types of storms, both of which occur regularly in Ohio. The dV_T equations would be more appropriate for estimating runoff volumes from frontal-type storms characterized by moderate to heavy rainfall of long duration, whereas the hydrograph method would be more appropriate for estimating runoff volumes from convective-type storms (thunderstorms) characterized by intense rainfall of average duration

SUMMARY

A study was conducted to develop methods to estimate peak-frequency relations, flood hydrographs, and volume-duration-frequency relations of small, ungaged urban streams in Ohio The methods were developed to assist planners in the design of hydraulic structures for which hydrograph routing may be required or where the temporary storage of water is an important element of the design criteria

The data base for the analyses consisted of 5-minute rainfall-runoff data collected for a period of 5–8 years at 62 small drainage basins located throughout Ohio. The U.S. Geological Survey rainfall-runoff model A634 was calibrated for each site The calibrated models were used in conjunction with long-term (66–87 years) rainfall and evaporation records to synthesize a long-term series of flood-hydrograph records at each site. A method was developed and used to increase the variance of the synthetic flood characteristics in order to make them more representative of observed flood characteristics


d or t
dV_T
VQ_T (t) Equation
Computation
$VQ_T(t)$
0
1
2
4
8
12
14
$0V_{100}$
$1V_{100}$
$2V_{100}$
$4V_{100}$
$8V_{100}$
EXPLANATION
DISCHARGE, IN CUBIC FEET PER SECOND
400
300
200
100
0
0
8
16
24
32
TIME, IN HOURS
1
2
4
8
16
32
d DURATION, IN HOURS
t TIME, IN HOURS
dV_T VOLUME, IN MILLIONS OF CUBIC FEET, OF d HOURS DURATION AND T YEARS RECURRENCE INTERVAL
$VQ_T(t)$ CUMULATIVE VOLUME, IN MILLIONS OF CUBIC FEET, OF t HOURS TIME AND T YEARS RECURRENCE INTERVAL
$VQ_{100}(0)$
$\frac{1}{2}$
$32V_{100}$
minus
$16V_{100}$
equals
$\frac{1}{2}$
equals
$VQ_{100}(8)$
$\frac{1}{2}$
$32V_{100}$
minus
$8V_{100}$
equals
$\frac{1}{2}$
equals
$VQ_{100}(12)$
$\frac{1}{2}$
$32V_{100}$
minus
$4V_{100}$
equals
$\frac{1}{2}$
equals
$VQ_{100}(14)$

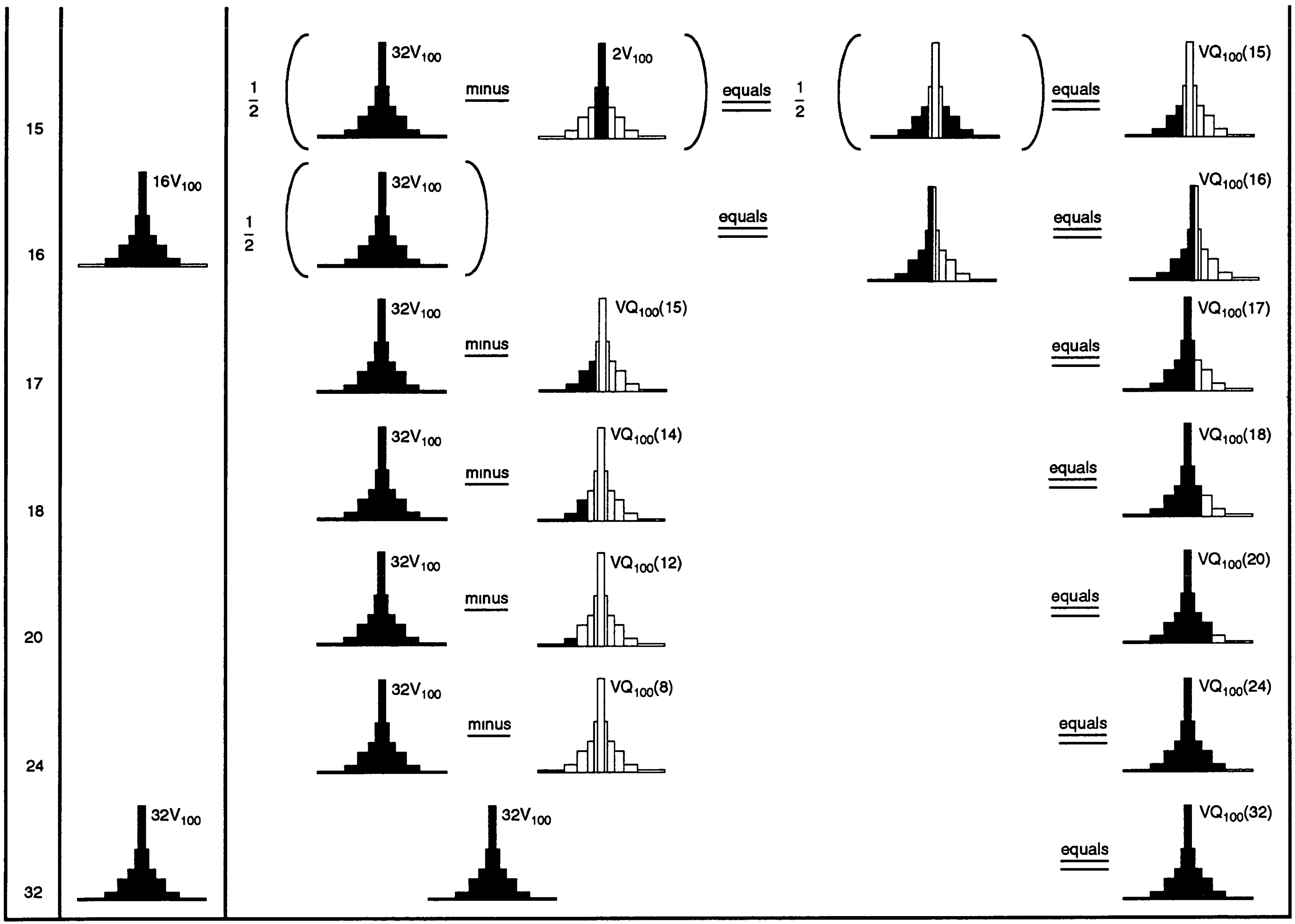


Figure 17. Illustration of a method to compute cumulative volume as a function of time ($VQ_T(t)$) from volume as a function of duration (dV_T)

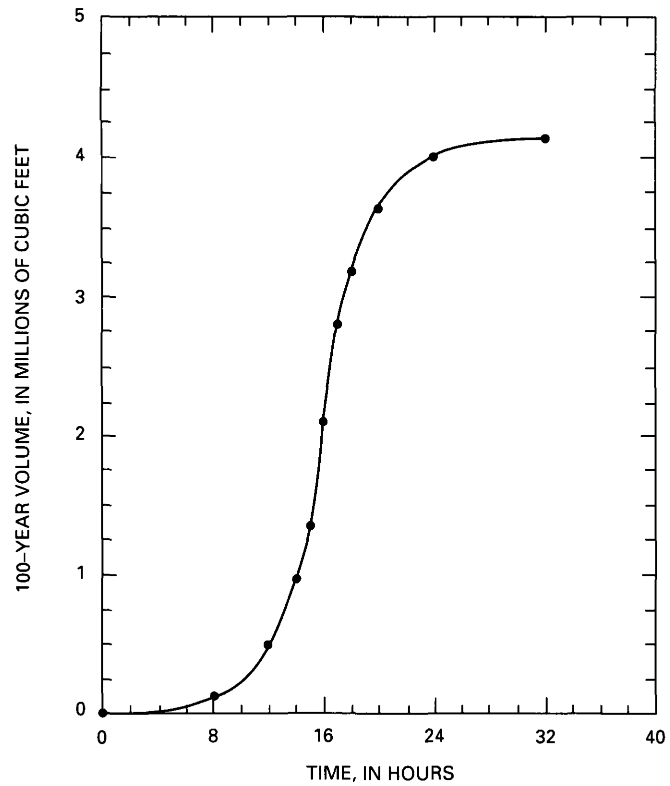


Figure 18. Estimated 100-year volumes as a function of time for an ungaged urban stream in Toledo, Ohio

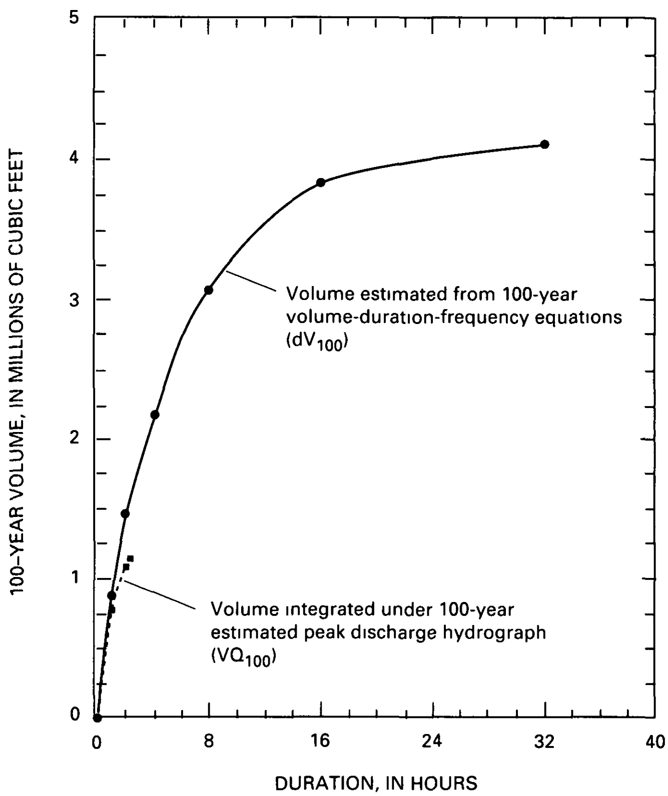


Figure 19 Volume estimated from 100-year volume-duration-frequency equations and volume integrated under 100-year estimated peak-discharge hydrograph for an ungaged urban stream in Toledo, Ohio

The logarithms of the annual peak discharges for each site were fit by a Pearson Type III frequency distribution to develop a peak-frequency relation for each site The peak-frequency data were related to various physical and climatic characteristics of 30 urban basins by multiple-regression analysis Multiple-regression equations were developed for estimating peak discharges having recurrence intervals of 2, 5, 10, 25, 50, and 100 years

The explanatory variables are drainage area, average annual precipitation, and basin-development factor Average standard errors of prediction for the peak-frequency equations range from ±34 to ±40 percent

A method was presented to estimate flood hydrographs by applying a specific peak discharge and an estimated basin lagtime to a dimensionless hydrograph An equation was developed to estimate basin lagtime in which main-channel length divided by the square root of the main-channel slope ($L/\sqrt{SL}$) and basin-development factor are the explanatory variables and the average standard error of prediction is ±53 percent A dimensionless hydrograph developed by the USGS for use in Georgia was verified for use in urban areas of Ohio

The largest runoff volume for each of six durations (1, 2, 4, 8, 16, and 32 hours) was computed for each water year of synthetic hydrograph data The logarithms of the annual peak volumes for each duration were fit by a Pearson Type III frequency distribution to develop a volume-duration-frequency relation for each site The volume-duration-frequency data were related to physical and climatic characteristics of 62 urban and rural basins by multiple-regression analysis Multiple-regression equations were developed for estimating maximum flood volumes of *d*-hour duration and *T*-year recurrence interval (dV_T) Flood-volume data for all combinations of six durations (1, 2, 4, 8, 16, and 32 hours) and six recurrence intervals (2, 5, 10, 25, 50, and 100 years) were analyzed The explanatory variables in the resulting 36 equations are drainage area, average annual precipitation, and basin-development factor Standard errors of prediction for the 36 dV_T equations range from ±28 to ±44 percent

Examples of how to use the methods are presented Volumes estimated by use of the volume-duration-frequency equations were compared with volumes estimated by integrating under an estimated hydrograph. Both methods yield similar results for volume estimates of short duration that are applicable to convective-type storm runoff The volume-duration-frequency equations can be used to compute volume estimates of long and short duration because the equations are based on maximum-annual-volume data of long and short duration The dimensionless-hydrograph method is based on flood hydrographs of average duration and cannot be used to compute volume estimates of long duration Volume estimates of long duration, which are applicable to runoff from frontal-type storms, may be considerably greater than volume estimates of short duration

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