TECHNIQUE FOR SIMULATING PEAK-FLOW HYDROGRAPHS IN MARYLAND

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 97-4279

Prepared in cooperation with the

MARYLAND STATE HIGHWAY ADMINISTRATION

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By Jonathan J. A. Dillow

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Baltimore, Maryland

1998

U.S. DEPARTMENT OF THE INTERIOR

BRUCE BABBITT, Secretary

U.S. GEOLOGICAL SURVEY

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CONVERSION FACTORS, ABBREVIATIONS, AND VERTICAL DATUM

Multiply	By	To obtain
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
square mile (mi^2)	2.590	square kilometer
foot per mile (ft/mi)	0.1894	meter per kilometer
cubic foot per second (ft3/s)	0.02832	cubic meter per second
inch (in.)	25.40	millimeter

GLOSSARY

Basin development factor: An index that quantifies improvements to the basin drainage system.

Centroid: The point at which any pair of lines dividing a drainage basin into equal halves will intersect.

Dimensionless hydrograph: A unit hydrograph derived by dividing the flow ordinates of the unit hydrograph by a selected peak flow, and the time ordinates by the basin lagtime.

Drainage area: The planar area of a basin enclosed by a drainage divide.

Duration: The length of time during which excess rainfall occurs.

Forest cover: The part of a drainage basin where the land use is defined as forest.

Hyetograph: A plot of rainfall depth as a function of time.

Impervious area: The surface area of a drainage basin impermeable to the infiltration of rainfall.

Lagtime: The time from the centroid of rainfall excess to the centroid of the runoff hydrograph **.**

Main channel slope: The slope of the main drainage channel between two points located 10 percent and 85 percent of the total main-channel length upstream from the point of interest. The total main channel length is calculated by extending the upper end of themain drainage channel to the drainage divide of the basin.

Rainfall excess: The volume of rainfall available for direct runoff, equal to the total rainfall minus interception, depression storage, and absorption.

Recurrence interval: The average interval of years during which a given peak discharge can normally be expected to be exceeded once.

Regression analysis: A procedure used to obtain a mathematical relation between a dependent variable and one or more independent variables valid over the range of available data.

Standard error of estimate: Standard error calculated with data used to develop the relation, reflecting the inability of the relation to provide estimates that match the observed data.

Standard error of prediction: Standard error calculated with data not used to develop the relation, reflecting the standard error of estimate and the inability of the observed data used to develop the relation to describe the parameter being observed.

Unit hydrograph: The direct runoff hydrograph resulting from a unit depth of excess rainfall generated uniformly over the drainage basin at a constant rate for an effective duration.

TECHNIQUE FOR SIMULATING PEAK-FLOW HYDROGRAPHS IN MARYLAND

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ABSTRACT

The efficient design and management of many bridges, culverts, embankments, and flood-protection structures may require the estimation of time-of-inundation and (or) storage of floodwater relating to such structures. These estimates can be made on the basis of information derived from the peak-flow hydrograph. Average peak-flow hydrographs corresponding to a peak discharge of specific recurrence interval can be simulated for drainage basins having drainage areas less than 500 square miles in Maryland, using a direct technique of known accuracy. The technique uses dimensionless hydrographs in conjunction with estimates of basin lagtime and instantaneous peak flow.

Ordinary least-squares regression analysis was used to develop an equation for estimating basin lagtime in Maryland. Drainage area, main channel slope, forest cover, and impervious area were determined to be the significant explanatory variables necessary to estimate average basin lagtime at the 95-percent confidence interval. Qualitative variables included in the equation adequately correct for geographic bias across the State. The average standard error of prediction associated with the equation is approximated as plus or minus (+/-) 37.6 percent. Volume correction factors may be applied to the basin lagtime on the basis of a comparison between actual and estimated hydrograph volumes prior to hydrograph simulation.

Three dimensionless hydrographs were developed and tested using data collected during 278 significant rainfall-runoff events at 81 stream-gaging stations distributed throughout Maryland and Delaware. The data represent a range of drainage area sizes and basin conditions.

The technique was verified by applying it to the simulation of 20 peak-flow events and comparing actual and simulated hydrograph widths at 50 and 75 percent of the observed peak-flow levels. The events chosen are considered extreme in that the average recurrence interval of the selected peak flows is 130 years. The average standard errors of prediction were +/- 61 and +/- 56 percent at the 50 and 75 percent of peak-flow hydrograph widths, respectively.

INTRODUCTION

The efficient design and management of many bridges, culverts, embankments, and flood-protection structures may require the estimation of time-of-inundation and (or) storage of floodwater relating to such structures. These estimates can be made on the basis of information derived from the peak-flow hydrograph. The U.S. Geological Survey (USGS), in cooperation with the Maryland State Highway Administration (MDSHA), developed a technique to simulate average peak-flow hydrographs corresponding to peak discharges of specific recurrence interval. The technique uses **dimensionless hydrographs**[1] in conjunction with estimates of basin **lagtime** and instantaneous peak flow.

Purpose and Scope

This report describes the results of a study to develop a technique for simulating peak-flow hydrographs for streams in Maryland, and provides an example of the application of the technique. The simulation technique can be used to estimate an average design hydrograph for sites at which flow data are not available. The average hydrograph will differ considerably from the actual hydrograph of any single runoff event. Further limitations to the application of the technique are also listed in the report.

The data used to conduct the study included basin characteristics and peak-flow hydrographs from 81 gaged streams in Maryland and Delaware, as well as rainfall data exhibiting a range of **durations** from 24 recording and 53 nonrecording rainfall gages in and near the selected gaged drainage basins. Data for 278 peak-flow events during the water years 1981 through 1993, inclusive, were selected for use in the study.

The equations for estimation of basin lagtime and volume correction factors presented in the report were derived using ordinary least-squares multiple **regression analysis** techniques. The dimensionless hydrographs presented here were developed from **unit hydrographs** computed using the method described by O' Donnell (1960).

The basin characteristics data used in the study are listed in the report. The streamflow and rainfall data used were too voluminous to be included, but are on file at the Maryland District office of the U.S. Geological Survey in Baltimore, Md. The rainfall data are also available from the National Climatic Data Center in Ashville, N. C.

Description of Study Area

Maryland lies between 37°53′ and 39°43′ north latitude and 75°04′ and 79°29′ west longitude (fig. 1). The State has an irregular shape that would fit on a rectangle 240 mi long (east-west) by 125 mi wide (north-south). Excluding the area covered by the Chesapeake Bay, the State has a total area of 10,577 mi^2, of which 9,891 mi^2 is land and 686 mi^2 is inland water. As seen in figure 1, the State is characterized by a diverse physiographic setting, ranging from the low-relief Coastal Plain to the mountainous Appalachian Plateaus. Land use across most of the State can be characterized as agricultural or forested, with urban development concentrated in the Piedmont.

Previous Investigations

There is no previously published technique for simulating peak-flow hydrographs that is exclusively applicable to Maryland streams. Techniques presented by Stricker and Sauer (1982) and U.S. Department of Agriculture (1972) are applicable nationwide. However, the methods of Stricker and Sauer are only applicable to urban streams, and the methods of the Natural Resources Conservation Service (formerly the Soil Conservation Service) were developed using data from relatively small drainage basins and are presented with no definition of the inherent error that may be expected in their results.

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[1]. Words in **bold** are defined in the Glossary.

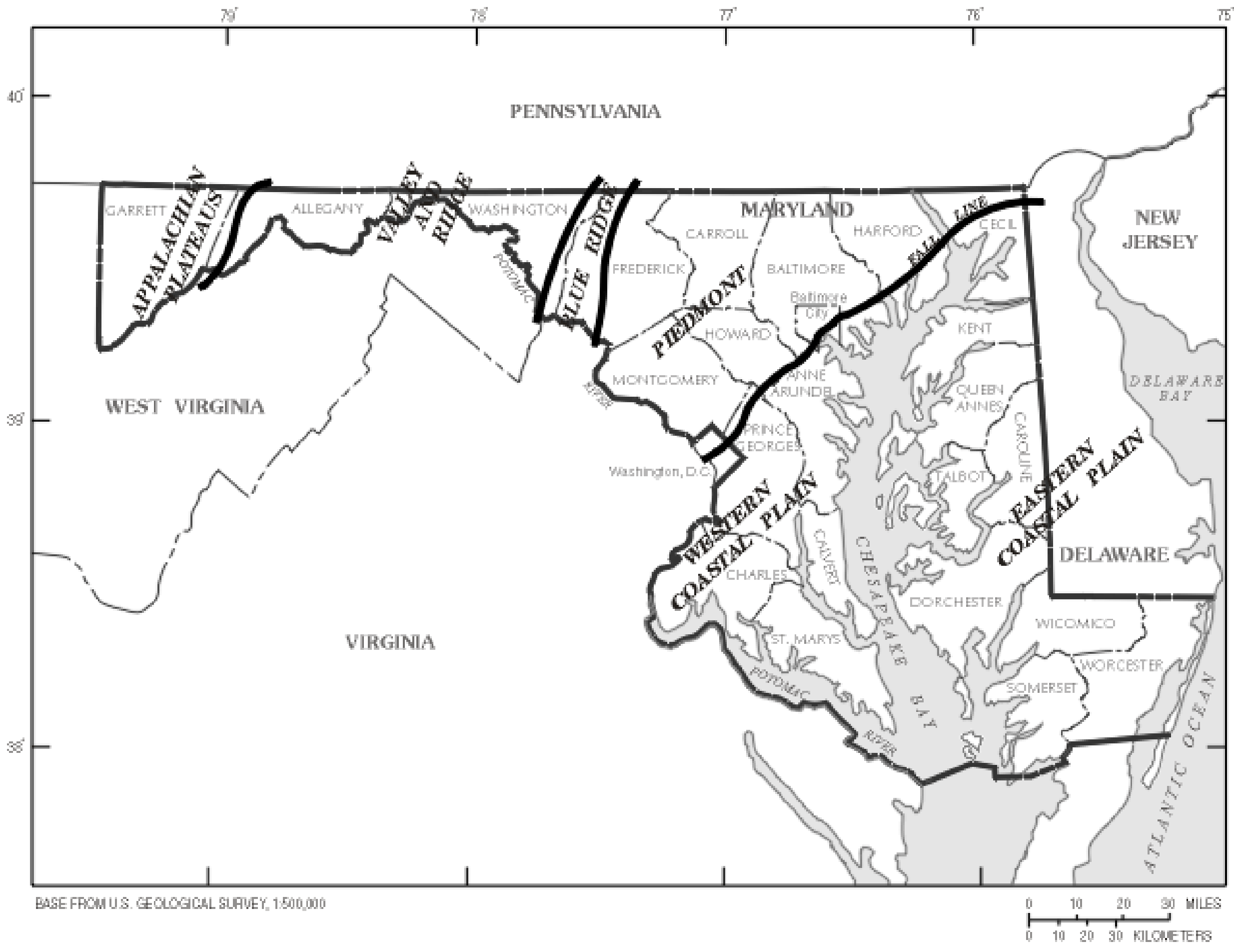


Figure 1. Study area and physiographic provinces in Maryland.

Studies by Inman (1986) in Georgia, and Bohman (1990) in South Carolina describe the methods used to conduct the study described in this report.

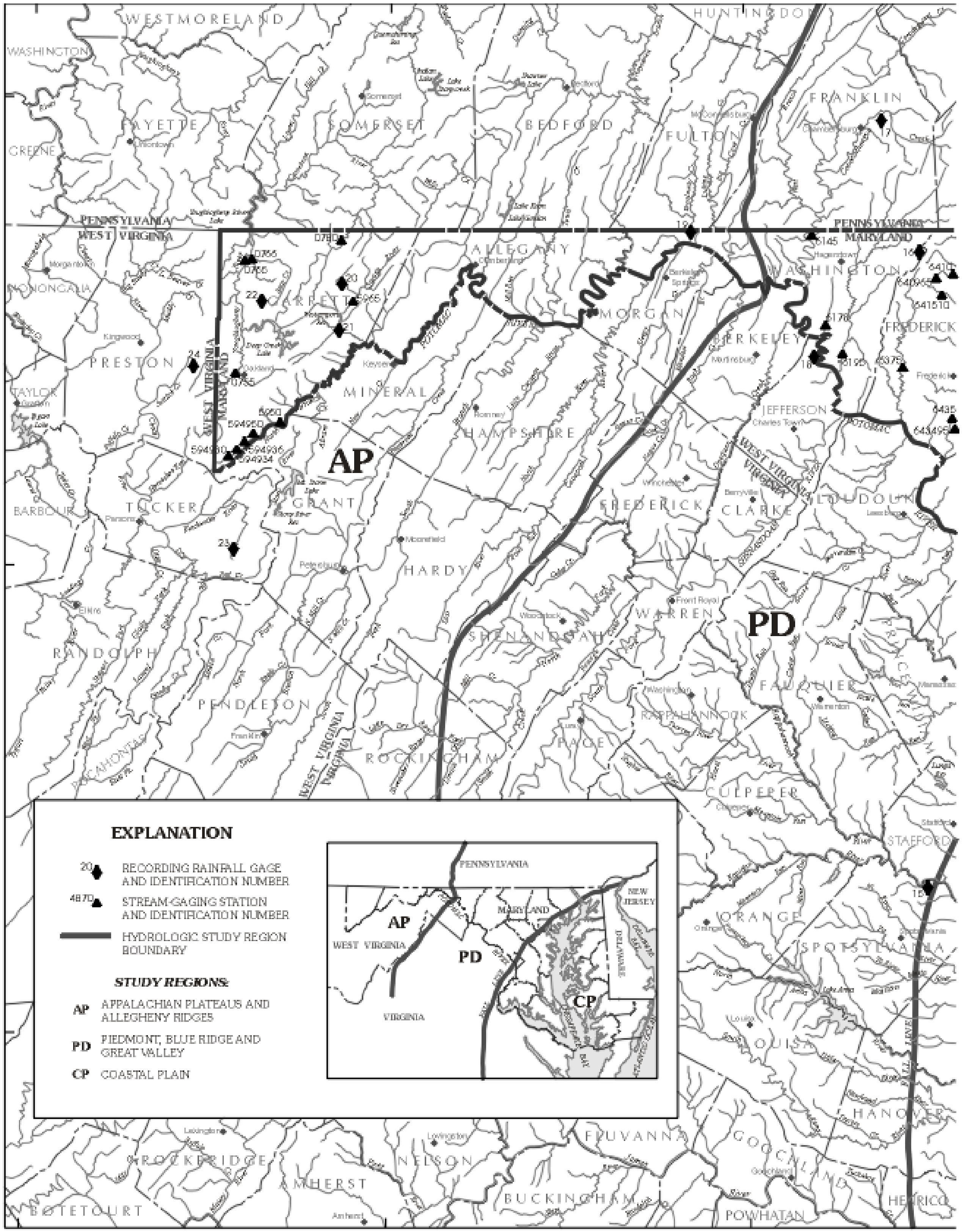
Acknowledgments

Special thanks is given to Wilbert O. Thomas who, in his capacity as advisor to the MDSHA, provided many valuable technical insights during the analysis phase of the study. Recognition is also due James R. Dine and Roger J. Starsonek of the Maryland-Delaware-District of Columbia District, and Patricia A. Kingrey of the Virginia District of the U.S. Geological Survey, whose work in retrieving and organizing archived rainfall and streamflow data made the timely completion of this report possible.

DESCRIPTION OF DATA BASE

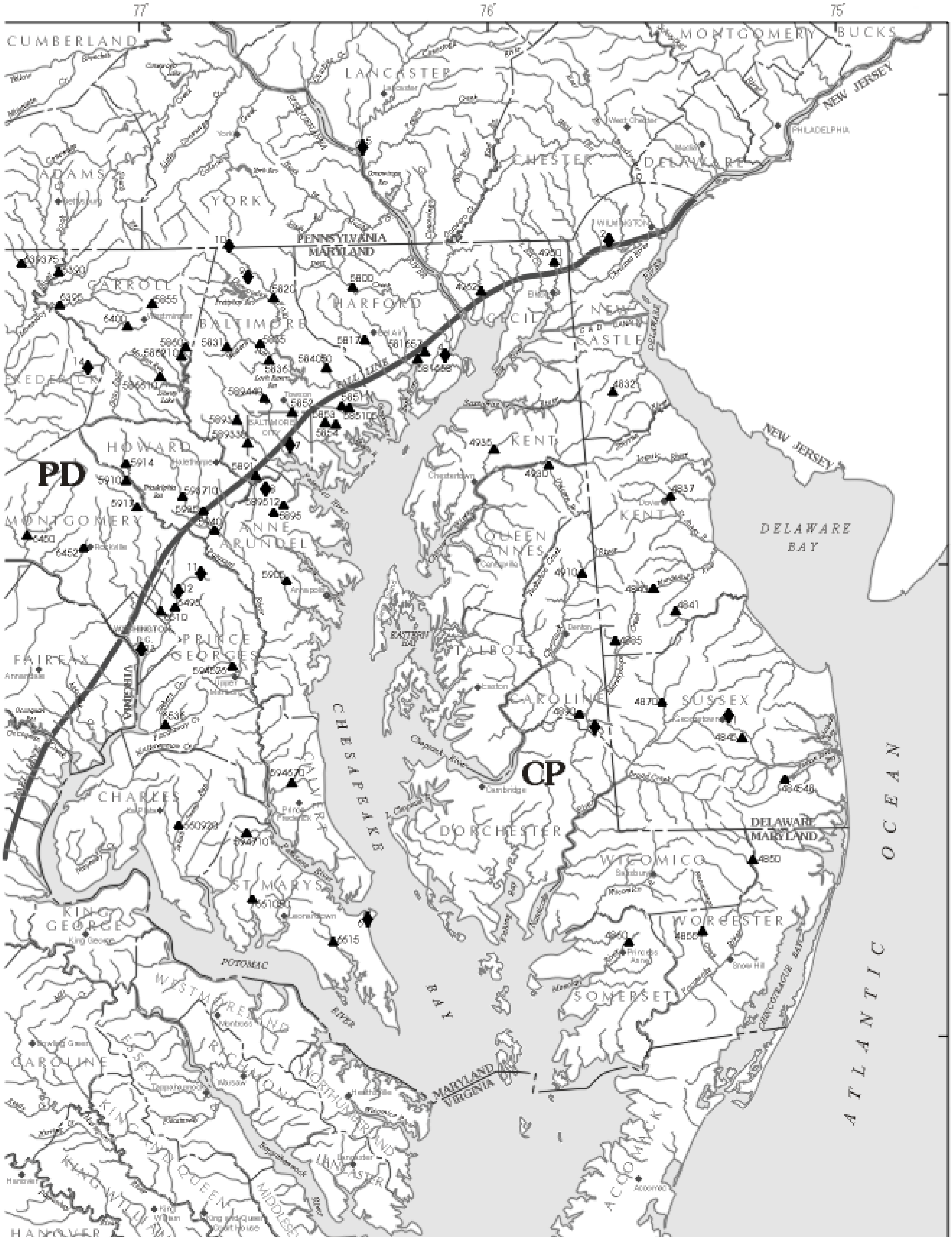
The data base used in this study consisted of information related to 278 rainfall-runoff events observed on 81 drainage basins in Maryland and Delaware for which streamflow and rainfall data were available (fig. 2), as well as selected physical characteristics associated with those basins.

The streamflow and rainfall data came from USGS continuous-record stream-gaging stations (table 1), and from the National Weather Service recording rainfall gages (table 2) and climatological stations, respectively. A data file containing rainfall and discharge data reported at



BASE FROM U.S. GEOLOGICAL SURVEY, 1:500,000, 1979

Figure 2. Location of recording rainfall gages, stream-gaging stations, and study regions in Maryland and surrounding States.


77°
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NEW JERSEY
PHILADELPHIA
ADAMS
YORK
CHESTER
DELAWARE
WILMINGTON
PENNSYLVANIA
MARYLAND
CARROLL
HARFORD
CECIL
BALTIMORE
NEW CASTLE
FREDERICK
HOWARD
KENT
PD
MONTGOMERY
ANNE ARUNDEL
QUEEN ANNES
DELAWARE BAY
FAIRFAX
PRINCE GEORGES
TALBOT
CAROLINE
SUSSEX
CHARLES
CALVERT
CHESAPEAKE BAY
CP
DORCHESTER
DELAWARE
MARYLAND
WICOMICO
ST MARYS
KING GEORGE
WORCESTER
POTOMAC RIVER
SOMERSET
ATLANTIC OCEAN
WESTMORELAND
CAROLINE
RICHMOND
NORTHUMBERLAND
MARYLAND
VIRGINIA
ESSEX
KING AND QUEEN
LANCASTER
ACCOMACK
KING WILLIAM
MIDDLESEX
HANOVER
0 10 20 30 40 MILES
0 10 20 30 40 KILOMETERS

Table 1. *Selected stream-gaging stations in Maryland and Delaware*

[°, degree; ′, minute; " , second]

Station no.	Station name	Latitude (° ′ ")	Longitude (° ′ ")	Station no.	Station name	Latitude (° ′ ")	Longitude (° ′ ")
01483200	Blackbird Creek at Blackbird, Del.	39 21 58	75 40 10	01496200	Principio Creek near Principio Furnace, Md.	39 37 34	76 02 27
01483700	St. Jones River at Dover, Del.	39 09 49	75 31 10	01580000	Deer Creek at Rocks, Md.	39 37 49	76 24 13
14840000	Murderkill River near Felton, Del.	38 58 33	75 34 03	01581657	Cranberry Run at Aberdeen, Md.	39 29 22	76 11 32
01484100	Beaverdam Branch at Houston, Del.	38 54 20	75 30 49	01581658	Cranberry Run at Perryman, Md.	39 28 42	76 12 08
01484500	Stockley Branch at Stockley, Del.	38 38 19	75 20 31	01581700	Winters Run near Benson, Md.	39 31 12	76 22 24
01484548	Vines Creek at Stockley, Del.	38 31 44	75 12 09	01582000	Little Falls at Blue Mount, Md.	39 36 16	76 37 16
01485000	Pocomoke River near Willards, Md.	38 23 20	75 19 30	01583100	Piney Run at Dover, Md.	39 31 15	76 46 02
01485500	Nassawango Creek near Snow Hill, Md.	38 13 44	75 28 19	01583500	Western Run at Western Run, Md.	39 30 38	76 40 37
01486000	Manokin Branch near Princess Anne, Md.	38 12 50	75 40 18	01583600	Beaverdam Run at Cockeysville, Md.	39 29 08	76 38 45
01487000	Nanticoke River near Bridgeville, Del.	38 43 42	75 33 44	01584050	Long Green Creek at Glen Arm, Md.	39 27 17	76 28 45
01488500	Marshyhope Creek near Adamsville, Del.	38 50 59	75 40 24	01585100	White Marsh Run at White Marsh, Md.	39 22 15	76 26 46
01489000	Faulkner Branch near Federalsburg, Md.	38 42 44	75 47 34	01585105	Honeygo Run at White Marsh, Md.	39 22 41	76 25 46
01491000	Choptank River near Greensboro, Md.	38 59 50	75 47 09	01585200	West Branch Herring Run at Idlewylde, Md.	39 22 25	76 35 05
01493000	Unicorn Branch near Millington, Md.	39 14 59	75 51 40	01585300	Stemmers Run at Rossville, Md.	39 20 28	76 29 17
01493500	Morgan Creek near Kennedyville, Md.	39 16 48	76 00 54	01585400	Brien Run at Stemmers Run, Md.	39 20 01	76 28 23
01495000	Big Elk Creek at Elk Mills, Md.	39 39 26	75 49 20	01585500	Cranberry Branch near Westminster, Md.	39 35 35	76 58 05

Table 1. *Selected stream-gaging stations in Maryland and Delaware*--Continued

Station no.	Station name	Latitude (° ′ ″)	Longitude (° ′ ″)	Station no.	Station name	Latitude (° ′ ″)	Longitude (° ′ ″)
01586000	North Branch Patapsco River at Cedarhurst, Md.	39 30 00	76 53 00	01594670	Hunting Creek near Huntingtown, Md.	38 35 02	76 36 20
01586210	Beaver Run near Finksburg, Md.	39 29 22	76 54 12	01594710	Killpeck Creek at Huntersville, Md.	38 28 37	76 44 08
01586610	Morgan Run near Louisville, Md.	39 27 07	76 57 20	01594930	Laurel Run at Dobbin Road near Wilson, Md.	39 14 37	79 25 43
01589100	East Branch Herbert Run at Arbutus, Md.	39 14 24	76 41 33	01594934	South Fork Sand Run near Wilson, Md.	39 15 29	79 25 07
01589300	Gwynns Falls at Villa Nova, Md.	39 20 45	76 44 01	01594936	North Fork Sand Run near Wilson, Md.	39 15 36	79 24 36
01589330	Dead Run at Franklintown, Md.	39 18 40	76 43 02	01594950	McMillan Fork near Fort Pendleton, Md.	39 16 36	79 23 26
01589440	Jones Fall at Sorrento, Md.	39 23 30	76 39 42	01595000	North Branch Potomac River at Steyer, Md.	39 18 07	79 18 26
01589500	Sawmill Creek at Glen Burnie, Md.	39 10 12	76 37 51	01596500	Savage River near Barton, Md.	39 34 05	79 06 10
01589512	Sawmill Creek at Crain Highway at Glen Burnie, Md.	39 10 59	76 36 51	01614500	Conococheague Creek at Fairview, Md.	39 42 57	77 49 28
01590500	Bacon Ridge Branch at Chesterfield, Md.	39 00 07	76 36 53	01617800	Marsh Run at Grimes, Md.	39 30 53	77 46 38
01591000	Patuxent River near Unity, Md.	39 14 18	77 03 23	01619500	Antietam Creek near Sharpsburg, Md.	39 27 01	77 43 52
01591400	Cattail Creek near Glenwood, Md.	39 15 21	77 03 05	01637500	Catoctin Creek near Middletown, Md.	39 25 35	77 33 25
01591700	Hawlings River near Sandy Spring, Md.	39 10 29	77 01 22	01639000	Monocacy River at Bridgeport, Md.	39 40 43	77 14 06
01593500	Little Patuxent River at Guilford Md.	39 10 04	76 51 07	01639375	Toms Creek at Emmitsburg, Md.	39 42 13	77 20 41
01593710	Middle Patuxent River near Simpsonville, Md.	39 11 48	76 53 59	01639500	Big Pipe Creek at Bruceville, Md.	39 36 45	77 14 10
01594000	Little Patuxent River at Savage, Md.	39 08 00	76 48 58	01640965	Hunting Creek near Foxville, Md.	39 37 10	77 28 00
01594526	Western Branch at Upper Marlboro, Md.	38 48 52	76 44 53	01641000	Hunting Creek at Jimtown, Md.	39 35 40	77 23 50

Table 1. *Selected stream-gaging stations in Maryland and Delaware*--Continued

Station no.	Station name	Latitude (° ′ ″)	Longitude (° ′ ″)	Station no.	Station name	Latitude (° ′ ″)	Longitude (° ′ ″)
01641510	Fishing Creek Tributary near Lewistown, Md.	39 33 09	77 26 48	01660920	Zekiah Swamp Run near Newtown, Md.	38 29 26	76 55 37
01643495	Bennett Creek Tributary at Park Mills, Md.	39 17 21	77 23 46	01661050	St. Clement Creek near Clements, Md.	38 20 00	76 43 31
01643500	Bennett Creek at Park Mills, Md.	39 17 40	77 24 30	01661500	St. Marys River at Great Mills, Md.	38 14 36	76 30 13
01645000	Seneca Creek at Dawsonville, Md.	39 07 41	77 20 13	03075500	Youghiogheny River near Oakland, Md.	39 25 19	79 25 32
01645200	Watts Branch at Rockville, Md.	39 05 03	77 10 38	03076500	Youghiogheny River at Friendsville, Md.	39 39 13	79 24 31
01649500	Northeast Branch Anacostia River at Riverdale, Md.	38 57 37	76 55 34	03076600	Bear Creek at Friendsville, Md.	39 39 22	79 23 41
01651000	Northwest Branch Anacostia River near Hyattsville, Md.	38 57 09	76 58 00	03078000	Casselman River at Grantsville, Md.	39 42 08	79 08 12
01653600	Piscataway Creek at Piscataway, Md.	38 42 20	76 58 00				

15-minute intervals was created for each event, after processing the rainfall data for **hyetograph** simulation, leading to 278 files describing the rainfall-runoff events.

The drainage area, main channel length, main channel slope, and storage of the drainage basins were obtained from data available in the USGS Basin Characteristics File of the Water Data Storage and Retrieval System (WATSTORE) as of 1993, or from the best available topographic maps, with the exceptions of **forest cover**, derived using 1972 land-use data from Alexander and others (1976), and **impervious area** and **basin development factor**, which were derived using 1990 land-use data from the Maryland Office of State Planning (1991) and field inspection, respectively. The 1972 land-use data was used to obtain forest cover because some of the basins in the study extended outside the boundaries of Maryland, and the 1990 land-use data does not cover areas outside the Maryland boundary.

Table 2. *Selected rainfall gages in Maryland, Delaware, Pennsylvania, Virginia, and West Virginia*

[°, degree; ′, minute]

Rainfall gage no.	Rainfall gage name	Latitude (° ′)	Longitude (° ′)	Period of record
1	Georgetown, Delaware	38 38	75 27	05/1971-12/1993
2	Newark University Farm, Delaware	39 40	75 44	06/1978-12/1993
3	Federalsburg, Maryland	38 41	75 46	06/1971-12/1993
4	Aberdeen (Phillips Field), Maryland	39 28	76 10	06/1979-12/1993
5	Safe Harbor Dam, Pennsylvania	39 55	76 23	01/1984-12/1993
6	Patuxent River, Maryland	38 20	76 25	04/1976-12/1993
7	Baltimore City, Maryland	39 17	76 37	01/1984-12/1993
8	BWI Airport, Maryland	39 11	76 40	01/1984-10/1993
9	Parkton, Maryland	39 38	76 42	09/1971-04/1987
10	Millers, Maryland	39 43	76 48	03/1988-12/1993
11	Beltsville, Maryland	39 02	76 53	05/1971-12/1993
12	College Park, Maryland	38 59	76 57	01/1984-12/1993
13	Washington, D.C., National Airport	38 51	77 02	01/1984-10/1993
14	Unionville, Maryland	39 27	77 11	04/1971-12/1993
15	Fredericksburg, Virginia	38 18	77 28	09/1978-03/1993
16	Catoctin Mountain Park, Maryland	39 39	77 29	05/1971-12/1993
17	Chambersburg, Pennsylvania	39 56	77 38	09/1971-12/1993
18	Shepherdstown, West Virginia	39 26	77 48	03/1979-12/1993
19	Hancock, Maryland	39 42	78 11	05/1975-12/1993
20	New Germany, Maryland	39 37	79 08	08/1978-05/1992
21	Savage River Dam, Maryland	39 31	79 08	04/1977-12/1993
22	McHenry, Maryland	39 35	79 22	10/1971-11/1993
23	Canaan Valley, West Virginia	39 03	79 26	10/1971-12/1993
24	Terra Alta, West Virginia	39 27	79 33	01/1984-12/1993

METHODS OF STUDY

The following sections explain the data-selection process and identify the methods of analysis used to conduct the study. The data required for the study included (1) unrestricted inflow, direct-runoff hydrographs, (2) the rainfall hyetograph associated with each direct-runoff hydrograph, and (3) the physical characteristics of each drainage basin providing direct-runoff hydrographs. The analytical methods used include multiple linear regression analysis and instantaneous unit hydrograph simulation, and result in estimates of basin lagtime and average dimensionless hydrograph shapes that can be used to simulate peak-flow hydrographs.

Stream-Gaging Station Selection

The most critical type of information needed to develop a hydrograph simulation technique was the direct-runoff hydrograph recorded at a continuous-record stream-gaging station. Any gaged stream that was affected by regulation during peak-flow events was automatically excluded from consideration. Because the study was focused on Maryland, to be included, a gaged stream had to have a drainage basin **centroid** either in the State or within 25 mi of the Maryland border.

Data from all continuous-record stream-gaging sites in and near Maryland that were active from water year [2] 1981 through water year 1993 were reviewed to determine the number of significant rainfall-runoff events for each site during that time period. The time period 1981-93 was chosen for analysis because the unit-values hydrograph data from more recent years are more easily accessible than older data. The methods to be used required that each site selected to provide data for the study had to exhibit at least three significant events. For this study, a significant event was arbitrarily defined as having an equivalent depth of rainfall of at least 0.25 in., where equivalent depth refers to the depth of water throughout the drainage basin if the total volume of rainfall occurred instantaneously and was evenly distributed over the basin. One event with less than 0.25 in., in equivalent depth of rainfall was included in the study. The event occurred in a highly developed basin with a strong runoff response to precipitation.

The 81 gaged sites selected were chosen according to the aforementioned constraints, and represent about two-thirds of the sites in the region, which constitutes a significant sample.

Hyetograph Development

Another type of information that was essential to the study was the rainfall hyetograph. Knowing both the shape and timing of the direct-runoff hydrograph and the rainfall hyetograph allowed the calculation of basin lagtimes for each rainfall-runoff event used in the study. Because rainfall data were not available at any of the selected stream-gage sites, a number of assumptions were used to derive the rainfall hyetograph for each event.

Rainfall was assumed to occur in a spatially uniform manner because of the relative sparsity of rainfall data. With that in mind, the magnitude of total rainfall for each event was calculated as the distance-weighted average of the rainfall reported at recording rain gages and climatological data stations reporting daily rainfall totals in the vicinity of each gaged drainage basin. The weighting method, as reported by Dean and Snyder (1977), uses the inverse of the squared distance between various rain gages and the centroid of a drainage basin to derive a weighted average of the rainfall data. The resulting rainfall estimates have accuracy comparable to other common rainfall-derivation methods.

The temporal distribution of rainfall reported by the nearest representative recording rain gage was assumed to characterize the temporal rainfall pattern for the hyetograph of each event. Using

[2]. The 12-month period October 1 through September 30, designated by the calendar year in which it ends.

these methods and assumptions, a rainfall hyetograph was derived for each event in the data base.

Predictor Variable Identification

The third type of information essential to the development of the technique was the physical basin characteristics of each drainage basin selected for study. The specific characteristics to be considered for use in the study were determined by reviewing the characteristics used in studies of this kind previously conducted in other States. The list of considered variables includes: **drainage area**, **main channel slope**, main channel length, sinuosity ratio, basin storage, basin shape factor, length to centroid, drainage density, forest cover, impervious area, and basin development factor. Length to centroid and drainage density were removed from consideration prior to analysis because of the relative difficulty associated with their measurement.

Analytical Methods

The equation for predicting basin lagtime and the equations used to calculate the volume correction factors were derived by multiple linear regression analysis as described by Riggs (1968), using the Statit (Statware, Inc., 1992) data analysis computer system. Minimization of the mean square error was used as the selection criterion for the best lagtime equation, and a 95-percent confidence limit was used to identify significant explanatory variables.

The dimensionless hydrographs were developed from unit hydrographs of selected events calculated using the method presented by O' Donnell (1960). The O' Donnell method utilizes the relation between **rainfall excess**, the instantaneous unit hydrograph, and the runoff hydrograph. The hyetograph of rainfall excess and the resultant runoff hydrograph can both be expressed by a sum of a harmonic series (sine and cosine functions). The unit hydrograph for a given basin is computed from the harmonic coefficients for the curves of rainfall excess and the runoff hydrograph. The method is codified in computer programs (S.E. Ryan, U.S. Geological Survey, written commun., 1986) that were used to perform hydrograph analysis for the study. These programs were specifically designed to aid in the development and testing of average dimensionless hydrographs.

The simulation technique is verified using the average **standard error of prediction** expressed as percentages of the simulated hydrograph widths at 50 and 75 percent of the peak flow exhibited by the hydrograph. These values were derived by comparing simulated hydrographs with 20 recorded hydrographs not used in developing the technique. Verifications of the accuracy of the basin lagtime equation and the dimensionless hydrograph shapes presented in the report are also expressed as average standard errors of prediction, derived from data as explained in the sections, ' Verification by Prediction Error Sum of Squares (PRESS)' and ' Shape Verification' , respectively.

DETERMINATION OF LAGTIME

Lagtime is defined as the time from the centroid of rainfall excess to the centroid of the resultant peak-flow hydrograph (Stricker and Sauer, 1982). Lagtime can be used in conjunction with a dimensionless hydrograph to predict peak-flow-hydrograph widths. Lagtime is considered to be a relatively constant value for a given drainage basin, dependent on the land use and other physical characteristics of the basin, and independent of the temporal and spatial distribution of rainfall.

Basin Lagtime

Basin lagtime is determined by finding the arithmetic average of the lagtimes associated with runoff events chosen to represent a basin in the data set. In this study, the lagtimes for the selected events were determined by the following procedure. The runoff hydrograph and rainfall hyetograph were obtained for each event. Base flow was removed from the runoff hydrograph by interpolating linearly between the first and last recorded discharge values of the hydrograph. The resulting volume of direct runoff was equated with the volume of excess rainfall. The rainfall hyetograph was then truncated by using a constant abstraction rate resulting in an excess rainfall hyetograph of the correct volume. The position of the centroids of both the excess rainfall hyetograph and the direct runoff hydrograph were computed, and the lagtime was calculated as the temporal

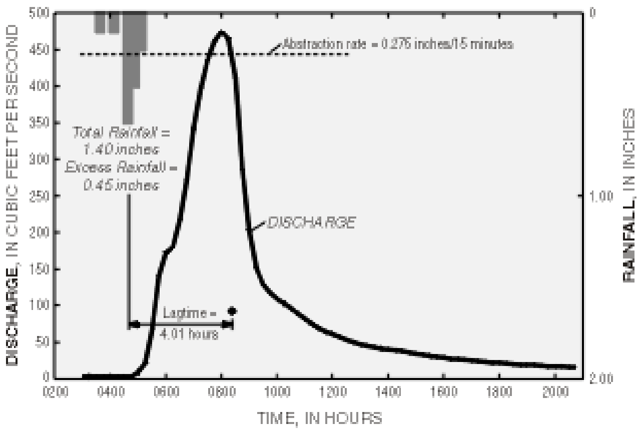

Figure 3. Abstraction rate and lagtime for Cranberry Run at Perryman, Maryland (station no. 01581658), July 20, 1989.

difference in the positions of the two centroids (fig. 3).

The basin lagtime was used to expand the dimensionless hydrograph time axis to simulate peak-flow hydrographs. This means that a hydrograph estimated using this technique will represent the average peak-flow hydrograph corresponding to a peak discharge of specific **recurrence interval** selected by the user. So that hydrograph estimates can be made for streams where no rainfall-runoff data have been collected, basin lagtime was related to selected basin characteristics by multiple linear regression analysis techniques.

Regional Analysis

Using multiple linear regression analysis techniques to relate basin lagtime to physical basin characteristics provides a means of estimating basin lagtime for any drainage basin in Maryland. Using the criteria already discussed to select the best lagtime prediction equation, the first analysis resulted in one equation to be used in all areas of the State. When compared to the calculated lagtimes, however, the estimates from the equation exhibited geographic bias.

Subsequent analyses were conducted on subsets of the statewide data base in an effort to define an estimating equation for each geographic area defined by the observed biases from the first analysis. Dividing the data base in this manner resulted in small data subsets for some regions that did not have sufficient degrees of freedom to support regression analysis. Degrees of freedom is the characteristic for a statistic for variation that is equivalent to the number of observed data values being used less the number of parameters being estimated.

To avoid the problem of insufficient degrees of freedom, it was decided that the data set would not be subdivided by geographic region, and that the geographic biases in the basin lagtime estimates would be addressed by adding qualitative variables to the equation. The addition of the qualitative variables allows the equation to adjust the basin lagtime estimate on the basis of the geographic location of the selected drainage basin. The study regions defined according to the areas of geographic bias--Appalachian Plateaus and Allegheny Ridges (AP), Piedmont, Blue Ridge and Great Valley (PD), and Coastal Plain (CP)--are shown in figure 2.

This approach produced one lagtime equation with a **standard error of estimate** of 34.9 (plus (+) 41 or minus (-) 29) percent, whose estimates exhibit no appreciable geographic bias:

$$LT = 0.18A^{0.234}\, SL^{-0.312}\, (101 - F)^{-0.220}\, (101 - IA)^{1.06}\, (10^{[0.219AP + 0.202\, CP]}) \quad (1)$$

where

LT is the basin lagtime, in hours;

A is the drainage area, in square miles;

SL is the main channel slope, in feet per mile;

F is the forest cover, in percent;

IA is the impervious area of the basin, in percent; and

AP, *CP* are qualitative variables with discrete values of 0 or 1. A value of one is assigned when the basin for which lagtime is being estimated is in the corresponding study region, as defined in figure 2.

The values of basin characteristics and basin lagtimes for 78 drainage basins in the data set used to develop the equation, as well as the basin characteristics of three drainage basins having undetermined basin lagtimes, are listed in table 3. Methods for determining each of the four independent basin characteristics used in equation 1 are addressed in the section, ‘ Simulating a Peak-Flow Hydrograph’ .

Testing of Lagtime Equation

Various tests were performed to ensure that estimates resulting from the use of equation 1 will be reliable within stated limits of accuracy when applied in accordance with the discussions presented in this report. As previously stated, any appreciable geographic bias was eliminated during equation development. Also, the standard error of estimate has been calculated as a measure of the accuracy of the equation with respect to the development data set. The standard error of estimate is a measure of the inability of the equation to provide basin lagtime estimates that match the observed basin lagtimes in the data set used to develop the equation.

Other tests and statistics used to define the accuracy and reliability of the equation include: the average standard error of prediction, as estimated by the prediction error sum of squares; variable bias testing; and tests of sensitivity to errors in variable measurement. The average standard error of prediction is a measure of the accuracy of the equation based on the inability of the equation to match estimates to observed data, and the inability of the observed data to describe the actual basin lagtimes of drainage basins. Note that the average standard error of prediction is always larger than the standard error of estimate.

Verification by Prediction Error Sum of Squares (PRESS)

Regression equations are usually verified by randomly splitting a data set in half, then using one half to develop the equation and the other half to verify it. Because the data set has only 78 gaged drainage basins, this method of verification was undesirable. Instead, the full data set was used to develop the equation, and the prediction error sum of squares (PRESS) was used to estimate average standard error of prediction for verification purposes.

As explained by Helsel and Hirsch (1992), for a data set with *n* values, and using the selected predictor variables, PRESS uses *n*-1 observations to develop an estimating equation, then estimates the value of the excluded observation. It then excludes a different observation, repeating the process for each observation. The resulting prediction errors are then squared and summed to give PRESS. Dividing PRESS by *n* gives a good estimate of the average standard error of prediction. For equation 1, the estimate of the average standard error of prediction was 37.6 (+ 45/ -31) percent. This represents the range of error within which roughly two-thirds of all estimated basin lagtimes will fall when equation 1 is used as specified in this report. This measure of equation accuracy is applicable only to basin lagtime estimates made for drainage basins having basin characteristics within the ranges defined by the data used in developing equation 1.

Variable Bias by Residual Plots

Variable-bias tests were done by plotting the residuals, defined as the difference between the calculated lagtime and the estimated lagtime, and each of the four independent predictor variables (drainage area, main channel slope, forest cover, and impervious area) for all gaged drainage basins used to develop the equation. Visual inspection of the plots showed that the residuals plotted as a random scatter for each predictor variable, indicating no tendency to overpredict or underpredict basin lagtime. This indicates that there is no variable bias in the equation within the range of predictor variables used in developing it, and that the variable transformations used in the equation are of the proper form to describe the relations of the predictor variables to basin lagtime.

Sensitivity to Variable Errors

The level of accuracy of an estimate of basin lagtime is dependent on the accuracy with which the predictor variables are measured or estimated. Errors of a specified magnitude, in percent, were introduced into each of the four predictor variables to show the effect on the estimate of basin lagtime. The results are shown in table 4.

Table 3. *Basin characteristics for selected drainage basins in Maryland and Delaware*

[mi^2, square miles; ft/mi, feet per mile; mi, miles; %, percent; hrs, hours; CP, Coastal Plain; PD, Piedmont, Blue Ridge and Great Valley; AP, Appalachian Plateaus and Allegheny Ridges; --, data not determined]

Station no.	Study region	Drainage area (mi^2)	Main channel slope (ft/mi)	Main channel length (mi)	Basin storage (%)	Forest cover (%)	Impervious area (%)	Basin develop-ment factor	Basin lagtime (hrs)
01483200	CP	3.85	15.8	3.5	1.298	45	0.38	0	7.37
01483700	CP	31.9	4.66	12.3	11.927	21	4.46	2	27.41
01484000	CP	13.6	6.26	5.9	.626	34	.33	0	21.04
01484100	CP	2.83	7.12	2.5	.000	43	.00	0	14.54
01484500	CP	5.24	4.87	4.4	.000	39	3.24	0	12.82
01484548	CP	13.6	4.39	7.9	26.055	33	1.13	0	24.28
01485000	CP	60.5	1.49	14.6	18.396	25	.08	0	28.58
01485500	CP	44.9	3.56	12.2	1.326	79	.30	0	37.21
01486000	CP	4.80	5.47	4.1	.000	57	--	0	--
01487000	CP	75.4	3.23	13.7	.000	40	.85	0	20.80
01488500	CP	44.8	2.65	11.7	.000	39	.14	0	12.99
01489000	CP	8.50	7.65	5.3	.000	24	.00	0	5.78
01491000	CP	113.	3.01	18.3	6.910	38	.66	0	31.57
01493000	CP	19.7	6.06	9.7	8.777	20	.35	0	26.10
01493500	CP	12.7	9.15	5.9	.199	5	.25	0	13.35
01495000	PD	52.6	17.9	22.2	.053	14	1.92	0	9.87
01496200	PD	9.03	29.0	5.9	.000	4	.00	0	4.38
01580000	PD	94.4	17.7	27.3	.039	27	.42	0	7.29
01581657	PD/CP	4.16	74.2	3.7	.000	33	5.25	0	4.08
01581658	PD/CP	5.22	56.1	4.8	.000	31	4.78	0	4.38
01581700	PD	34.8	30.0	15.8	.000	21	2.37	2	4.68
01582000	PD	52.9	33.8	15.0	.015	32	.91	0	6.84
01583100	PD	12.3	50.9	7.8	.092	26	.29	0	5.77
01583500	PD	59.8	24.5	15.9	.064	22	.16	0	8.20
01583600	PD	20.9	52.0	8.2	.309	29	18.6	4	5.63
01584050	PD	9.40	70.0	4.8	.000	13	1.00	0	3.05
01585100	PD/CP	7.61	48.2	6.0	.000	28	27.5	7	2.11
01585105	PD	2.65	65.2	3.6	.000	16	5.22	0	3.86
01585200	PD	2.13	72.7	2.2	.000	7	33.0	8	1.02

Table 3. *Basin characteristics for selected drainage basins in Maryland and Delaware--Continued*

Station no.	Study region	Drainage area (mi^2)	Main channel slope (ft/mi)	Main channel length (mi)	Basin storage (%)	Forest cover (%)	Impervious area (%)	Basin develop-ment factor	Basin lagtime (hrs)
01585300	PD/CP	4.46	54.7	4.6	0.558	28	23.6	6	2.06
01585400	CP	1.97	27.1	2.0	.000	24	35.1	2	2.33
01585500	PD	3.29	56.0	3.5	1.165	21	.45	0	3.08
01586000	PD	56.6	28.5	14.6	.069	19	1.77	0	8.56
01586210	PD	14.0	44.0	8.1	.000	19	1.77	0	4.39
01586610	PD	28.0	30.9	10.0	.000	20	.38	0	5.97
01589100	PD	2.47	87.1	3.2	.000	19	37.0	4	1.67
01589300	PD	32.5	21.0	13.7	.000	31	18.6	4	3.95
01589330	PD	5.52	52.1	3.2	.000	4	40.8	12	2.26
01589440	PD	25.2	38.2	9.5	.000	34	9.92	2	5.29
01589500	CP	4.97	24.8	4.4	.000	44	21.9	3	8.19
01589512	CP	8.24	23.5	5.9	1.092	31	30.8	3	6.72
01590500	CP	6.92	19.8	4.7	.000	65	1.87	0	10.90
01591000	PD	34.8	28.2	12.2	.000	21	.21	0	6.51
01591400	PD	22.9	28.0	8.7	.097	16	1.52	0	6.16
01591700	PD	27.0	26.5	10.9	.141	19	2.08	0	5.28
01593500	PD	38.0	15.8	15.5	.623	23	18.7	6	7.48
01593710	PD	48.4	17.8	14.7	.000	24	2.16	0	5.99
01594000	PD	98.4	13.6	23.5	.134	26	6.52	4	10.83
01594526	CP	89.7	8.2	16.1	.037	30	7.84	4	23.16
01594670	CP	9.38	16.9	5.2	.000	70	3.85	0	9.17
01594710	CP	3.26	41.8	2.9	.000	52	9.24	0	3.86
01594930	AP	8.23	26.4	4.4	.000	86	.00	0	7.50
01594934	AP	1.55	161.9	2.1	.000	82	.00	0	6.43
01594936	AP	1.91	130.9	2.7	.000	87	.00	0	6.62
01594950	AP	2.30	194.6	2.7	.000	89	.00	0	6.74
01595000	AP	73.0	30.5	16.5	.186	78	.49	0	12.27
01596500	AP	49.1	65.1	19.0	.066	80	.06	0	13.97
01614500	PD	494.	11.2	69.5	.101	37	1.43	0	25.42
01617800	PD	18.9	23.8	9.4	.000	2	2.32	0	15.53

Table 3. *Basin characteristics for selected drainage basins in Maryland and Delaware--Continued*

Station no.	Study region	Drainage area (mi^2)	Main channel slope (ft/mi)	Main channel length (mi)	Basin storage (%)	Forest cover (%)	Impervious area (%)	Basin develop-ment factor	Basin lagtime (hrs)
01619500	PD	281.	10.8	49.9	0.123	30	2.67	0	24.66
01637500	PD	66.9	47.5	23.3	.000	38	1.01	0	8.98
01639000	PD	173.	18.9	30.8	.114	20	.69	0	15.91
01639375	PD	41.3	75.4	12.2	.207	70	.87	0	3.47
01639500	PD	102.	13.5	26.9	.000	14	.13	0	11.80
01640965	PD	2.14	336.4	2.2	.000	92	.00	0	1.78
01641000	PD	18.4	145.2	9.7	.373	80	1.93	1	5.11
01641510	PD	.40	817.8	.9	.000	100	.00	0	--
01643495	PD	.15	1,000.	.5	.000	100	.00	0	--
01643500	PD	62.8	24.8	15.6	.000	23	1.19	0	7.30
01645000	PD	101.	14.0	21.2	.120	25	3.15	4	10.88
01645200	PD	3.70	67.4	2.7	.000	14	28.0	6	1.91
01649500	CP/PD	72.8	27.2	15.3	.192	33	22.0	5	8.85
01651000	PD/CP	49.4	19.7	19.1	.047	19	22.0	6	6.45
01653600	CP	39.5	16.1	14.4	.176	38	8.25	2	17.29
01660920	CP	79.9	10.6	16.6	5.051	56	3.60	0	26.17
01661050	CP	18.5	12.4	7.2	.000	56	3.09	0	14.26
01661500	CP	24.0	12.9	8.0	.000	78	2.46	0	15.78
03075500	AP	134.	6.09	19.3	.493	54	.88	0	22.57
03076500	AP	295.	22.2	40.8	3.180	66	.24	0	25.10
03076600	AP	48.9	65.6	15.3	.000	62	1.25	0	16.47
03078000	AP	62.5	28.2	19.5	1.005	75	.66	0	16.88

Table 4. *Sensitivity of computed basin lagtime to errors in variable measurement*

[*A*, drainage area; *SL*, main channel slope; *F*, forest cover, in percent; *IA*, impervious area, in percent; --, not determined]

Percent error in variable	Percent error in computed basin lagtime for indicated variable							
	A	*SL*	*F*			*IA*		
			10	50	90	10	30	50
-50	-15.0	24.1	-1.2	-8.4	-30.1	5.8	22.5	52.6
-25	-6.5	9.4	-.6	-4.7	-21.7	2.9	11.2	26.2
-10	-2.4	3.3	-.2	-2.0	-12.3	1.2	4.5	10.4
10	2.3	-2.9	.2	2.3	45.5	-1.2	-4.5	-10.4
25	5.4	-6.7	.6	6.4	--	-2.9	-11.2	-25.8
50	10.0	-11.9	1.3	16.0	--	-5.8	-22.2	-51.0

Note that because the two qualitative variables are discrete and of known value, they introduce no error into the equation because of measurement errors. Also note that because forest cover and impervious area are in the equation in the forms (101-*F*) and (101-*IA*), respectively, the initial magnitude of the variable estimate will affect the magnitude of error in estimated basin lagtime resulting from various levels of measurement error. To account for this, errors in the lagtime estimate were calculated for a range of assumed measurement errors as well as a range of initial measured values for these two predictor variables.

DEVELOPMENT OF DIMENSIONLESS HYDROGRAPHS

A dimensionless hydrograph can be expanded into a peak-flow hydrograph using the appropriate values of time and discharge. Data for 278 rainfall-runoff events occurring at 81 stream-gaging stations located throughout Maryland and Delaware were available for use in developing dimensionless hydrographs. Average dimensionless hydrographs were developed on the basis of data from 205 rainfall-runoff events at 62 stream-gaging stations, with peak-flow recurrence intervals ranging from 1.1 to 50 years, and mean and median recurrence intervals of 3.2 years and less than 2 years, respectively. The total rainfalls for these events ranged from 0.09 to 5.58 in. with a mean of 1.59 in., and the excess rainfall ranged from 0.02 to 3.31 in. with a mean of 0.53 in. The remaining 73 events at 19 stream-gaging stations, with peak-flow recurrence intervals ranging from 1.1 to 20 years, and mean and median recurrence intervals of 3.4 years and less than 2 years, respectively, were used to verify the shape of the derived dimensionless hydrograph. The verification events had total rainfalls ranging from 0.27 to 6.04 in. with a mean of 1.75 in., and the excess rainfall ranged from 0.07 to 1.68 in. with a mean of 0.56 in. All events included in the data base occurred during the 13-year period starting at the beginning of the 1981 water year and ending at the end of the 1993 water year.

The method used in selecting these events is defined in this report in the section, ‘ Stream-Gaging Station Selection’ . The analyses used to define the dimensionless hydrographs in this report were accomplished by use of a series of computer programs. The following steps, based on information from Inman (1986) and Bohman (1990), describe the procedure used in developing the dimensionless hydrographs:

(1) Compute a unit hydrograph and lagtime for three to five rainfall-runoff events for each of the 62 gaged streams in the data base that are not designated for use in verifying the dimensionless hydrographs using the unit hydrograph computation method described by O' Donnell (1960). Examples of a direct-runoff hydrograph with hyetograph and the corresponding unit hydrograph are shown in figures 4 and 5.

(2) Exclude any unit hydrographs with irregular shapes, including multiple peaks, from further use in hydrograph development.

(3) Compute an average unit hydrograph for each gaged stream by aligning the peaks and averaging each ordinate of discharge for the selected unit hydrographs. The position of the centroid of the average unit hydrograph is obtained by arithmetically averaging the positions of the centroids of the unaligned unit hydrographs. The results of this step for a typical gaged stream can be seen in table 5 and figure 6.

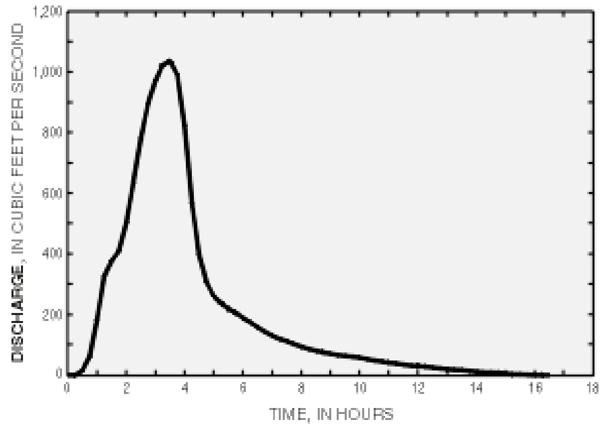

Figure 5. Unit hydrograph computed from observed data in figure 4.

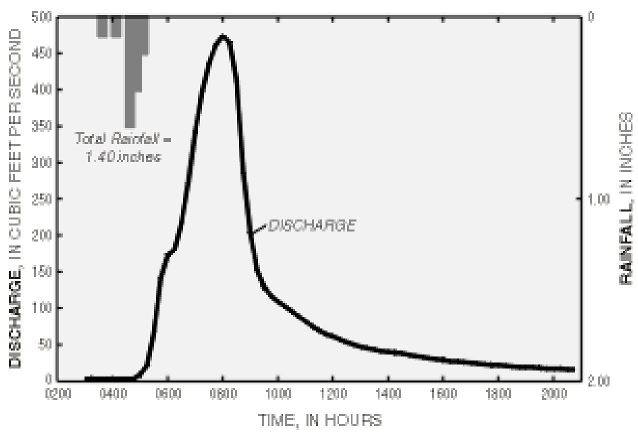

Figure 4. Observed peak-flow hydrograph and unit precipitation from Cranberry Run at Perryman, Maryland (station no. 01581658), July 20, 1989.

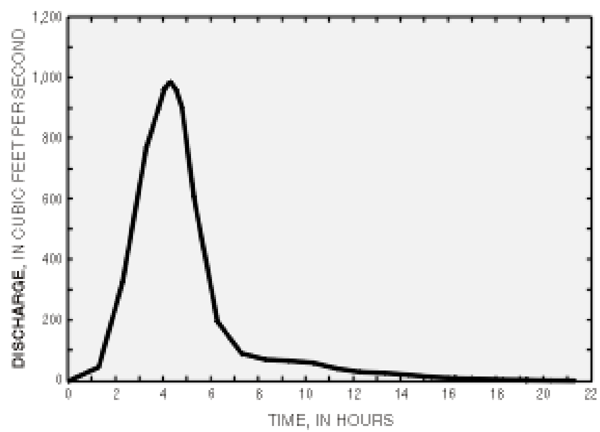

Figure 6. Average unit hydrograph for Cranberry Run at Perryman, Maryland.

Table 5. *Discharge ordinates at 1-hour intervals for four unit hydrographs and average unit hydrograph for Cranberry Run at Perryman, Maryland*

Discharge ordinates (cubic feet per second)				Average unit hydro-graph
09/13/1987	11/28/1988	05/06/1989	07/20/1989	
0	0	0	0	0
27	25	119	16	47
428	253	250	378	327
955	729	611	781	769
1,267	861	787	1,036	988
676	643	706	398	606
118	131	310	220	195
31	32	145	158	91
1	68	104	112	71
0	113	75	80	67
0	124	56	64	61
0	78	41	49	42
0	56	30	35	30
0	62	22	25	27
0	53	15	17	21
0	38	9	9	14
0	31	5	3	10
0	28	2	0	7
0	22	0	0	5
0	17	0	0	4
0	10	0	0	2
0	5	0	0	1

(4) Transform the average unit hydrographs computed in step 3 to hydrographs having durations of one-fourth, one-third, one-half, and three-fourths lagtime. These durations must be to the nearest multiple of the original duration (computation interval), which, for data used in this study, was 15 minutes. So that if the lagtime of an average unit hydrograph is 2.10 hours (126 minutes), the one-fourth lagtime is 31.5 minutes, which would be rounded to 30 minutes. One-third lagtime is 42 minutes, which would be rounded to 45 minutes. One-half lagtime is 63 minutes, which would be rounded to 60 minutes. Three-fourths lagtime is 94.5 minutes, which would be rounded to 90 minutes. The transformed unit hydrographs will have durations of 2-times, 3-times, 4-times, and 6-times the duration of the original average unit hydrograph. The transformation of a short duration unit hydrograph to a long-duration unit hydrograph can be accomplished through the use of the following equations:

$D/\Delta t$	EQUATION
2	$TUHD(t) = 1/2[TUH(t)+TUH(t-1)]$,
3	$TUHD(t) = 1/3[TUH(t)+TUH(t-1)+TUH(t-2)]$,
4	$TUHD(t) = 1/4[TUH(t)+TUH(t-1)+TUH(t-2)+TUH(t-3)]$, and
n	$TUHD(t) = 1/n[TUH(t)+TUH(t-1)+...+TUH(t-n+1)]$, (2)

where

Δt is the computation interval, equal to the duration of the original unit hydrograph,

D is the design duration of the unit hydrograph, a multiple of Δt,

$TUHD(t)$ are the ordinates of the desired unit hydrograph at time t, and

$TUH(t), TUH(t-1),..., TUH(t-n+1)$ are the ordinates of the original unit hydrograph at times $t, t-1,..., t-n+1$.

Duration may be thought of as actual duration or design duration, so a distinction must be made between the two. Actual duration, which is highly variable, is defined as the time during which precipitation falls at a rate greater than the existing infiltration capacity. It is the actual time during which rainfall excess is occurring. Design duration is that duration that is most convenient for use on any particular basin. The design duration is that for which the unit hydro-graph is computed. For this report, design duration is expressed as a fractional part of lagtime, such as one-fourth, one-third, one-half, and three-fourths lagtime.

(5) Reduce the one-fourth, one-third, one-half, and three-fourths lagtime hydrographs to dimensionless terms by dividing the time by basin lagtime and the discharge by peak discharge. The results of this step are shown in figure 7.

Having accomplished steps 1 through 5 for the 205 selected rainfall-runoff events, the final step in developing the average dimensionless hydrographs used in this technique was to regionalize and average the dimensionless hydrographs derived in step 5.

Regionalization of Dimensionless Hydrographs

The statewide average one-fourth, one-third, one-half, and three-fourths-lagtime duration dimensionless hydrographs were computed using the dimensionless hydrographs derived in step 5. A comparison of widths at 50 and 75 percent of the peak flow was made between hydrographs that were simulated using each of the four statewide hydrographs of various duration and the 205 observed peak-flow hydrographs used in their development. The results showed that all four statewide hydrographs had a strong tendency to underpredict actual hydrograph widths in the Appalachian Plateaus and Allegheny Ridges and in the Coastal Plain.

On the basis of this information, the statewide dimensionless hydrographs were discarded in favor of regional dimensionless hydrographs corresponding to the Appalachian Plateaus and Allegheny Ridges (AP), the Piedmont, Blue Ridge and Great Valley (PD), and the Coastal Plain (CP). The regional dimensionless hydrographs were developed by calculating the average one-fourth, one-third, one-half, and three-fourths-lagtime duration hydrographs for each of the three regions identified in the bias analysis of the statewide hydrographs.

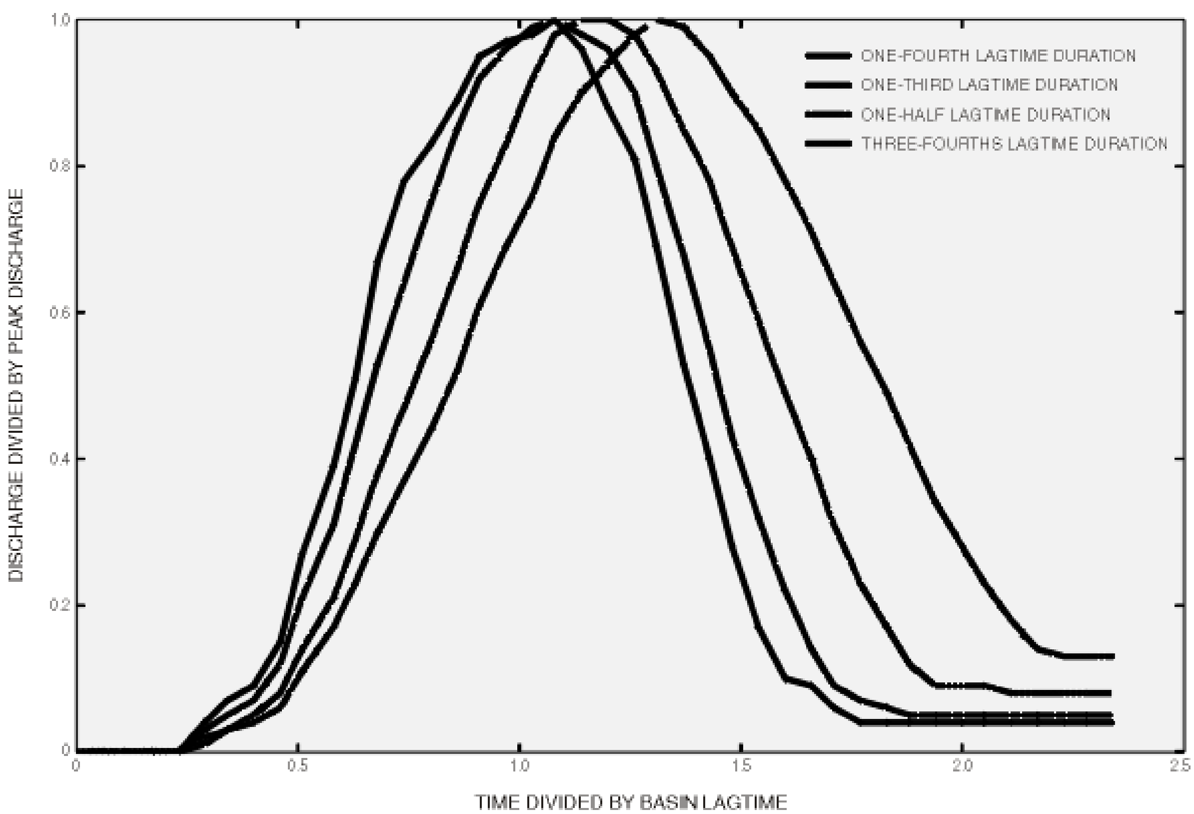

Figure 7. One-fourth, one-third, one-half, and three-fourths-lagtime duration dimensionless hydrographs for Cranberry Run at Perryman, Maryland.

The shape of a hydrograph can be defined by seven coordinates: the four points defining hydrograph width at 50 and 75 percent of the peak flow, along with the point defining the peak and the two end points. The dimensionless hydrograph shapes were checked by making a statistical comparison of the widths at 50 and 75 percent of peak flow between observed hydrographs used to develop the dimensionless hydrographs and hydrographs simulated from the three average dimensionless hydrographs.

The standard error of estimate for both of the width parameters in all three regions was found to be smallest when using hydrographs simulated from the three-fourths-lagtime duration hydrograph. The standard errors of estimate of hydrograph width, calculated using data from 205 observed peak-flow hydrographs used in dimensionless hydrograph development, were: +/- 47 and +/- 48 percent in the AP region; +/- 48 and +/- 50 percent in the PD region; and +/- 41 and +/- 43 percent in the CP region, for hydrograph widths at 50 and 75 percent of peak flow, respectively.

These results indicate that the three-fourths-lagtime duration hydrographs provide the best fitting design duration in all three regions and were chosen for use in hydrograph simulation. The coordinates of the three average dimensionless hydrographs are shown in table 6. The average dimensionless hydrographs to be used in Maryland and the Georgia dimensionless hydrograph developed by Inman (1986) and the U.S. Soil Conservation Service (SCS) dimensionless hydrograph developed by the U.S. Department of Agriculture (1972) are compared in figure 8.

Testing of Dimensionless Hydrographs

Several tests were performed to determine the validity of the shapes of the dimensionless hydrographs. The first test compared the widths of simulated hydrographs developed using the dimensionless hydrographs to the widths of the observed hydrographs used to develop the dimensionless hydrographs. The test results were reported in the preceding section as the standard errors of estimate. Other tests address verification, bias, and sensitivity.

Shape Verification

The validity of using the dimensionless hydrograph shapes to simulate peak-flow hydrographs in Maryland can be further assessed by verifying the results of the width comparison. This verification is done by performing width comparisons between simulated hydrographs and observed hydrographs not used in developing the dimensionless hydrographs. Because the dimensionless hydrographs are independent of the observed data, the accuracy of the hydrograph widths at 50 and 75 percent of peak flow determined by this test can be considered characteristic of the results that may be expected when applying the simulation technique.

This test was performed using data collected during 73 rainfall-runoff events at 19 stream-gaging stations distributed throughout the State. The resulting average standard errors of prediction for hydrograph widths at 50 and 75 percent of peak flow were: +/- 40 and +/- 37 percent in the AP region, +/- 61 and +/- 66 percent in the PD region, and +/- 29 and +/- 42 percent in the CP region, respectively.

Errors computed for model verification are generally expected to be greater than those computed when checking the model. This is not the case in the AP and CP regions. These results provide some confidence, however, that errors in simulated hydrograph width will not be significantly greater than those reported by comparison with the data used in developing the dimensionless hydrographs. In the PD region, the verification errors should be considered indicative of the simulated hydrograph width accuracy in that region.

Geographical and Width Bias

The dimensionless hydrographs were also tested for bias. Geographical bias was tested by plotting the positive and negative residuals of simulated and observed hydrograph widths at 50 and 75 percent of peak flow on a map. The map was then visually inspected to identify any areas in each of the three study regions in which hydrograph widths were being consistently over-predicted or underpredicted. No geographical bias was found.

Table 6. *Time and discharge ratios for the Appalachian Plateaus and Allegheny Ridges, Piedmont, Blue Ridge and Great Valley, and Coastal Plain regional dimensionless hydrographs*

[*t*, time; *LT*, basin lagtime; *Q*, discharge; Q_p, peak discharge; AP, Appalachian Plateaus and Allegheny Ridges; PD, Piedmont, Blue Ridge and Great Valley; CP, Coastal Plain; --, not available]

Time ratio (*t/LT*)	Discharge ratio (Q/Q_p) AP	PD	CP	Time ratio (*t/LT*)	Discharge ratio (Q/Q_p) AP	PD	CP
0.05	--	--	0.06	1.05	0.99	0.97	0.95
.10	--	--	.08	1.10	1.00	.99	.97
.15	--	--	.10	1.15	.99	1.00	.99
.20	0.05	--	.12	1.20	.97	.98	1.00
.25	.07	0.06	.14	1.25	.94	.96	.99
.30	.11	.08	.17	1.30	.89	.92	.97
.35	.15	.11	.19	1.35	.84	.86	.94
.40	.20	.14	.23	1.40	.79	.80	.90
.45	.26	.19	.27	1.45	.74	.74	.85
.50	.33	.25	.32	1.50	.68	.68	.81
.55	.41	.32	.38	1.55	.63	.61	.76
.60	.49	.40	.45	1.60	.58	.55	.72
.65	.57	.48	.53	1.65	.54	.50	.68
.70	.64	.56	.60	1.70	.49	.45	.63
.75	.71	.64	.67	1.75	.46	.41	.59
.80	.78	.72	.73	1.80	.42	.37	.55
.85	.84	.79	.78	1.85	.39	.33	.52
.90	.89	.85	.83	1.90	.36	.30	.48
.95	.94	.90	.88	1.95	.33	.28	.44
1.00	.97	.94	.91	2.00	.31	.25	.41

Table 6. *Time and discharge ratios for the Appalachian Plateaus and Allegheny Ridges, Piedmont, Blue Ridge and Great Valley, and Coastal Plain regional dimensionless hydrographs--Continued*

Time ratio (t/LT)	Discharge ratio (Q/Q_p) AP	PD	CP	Time ratio (t/LT)	Discharge ratio (Q/Q_p) AP	PD	CP
2.05	0.28	0.23	0.38	2.55	0.15	0.11	0.14
2.10	.27	.22	.35	2.60	.15	.10	.12
2.15	.25	.20	.32	2.65	.14	.10	--
2.20	.23	.19	.29	2.70	.13	.09	--
2.25	.21	.18	.27	2.75	.13	.08	--
2.30	.20	.16	.24	2.80	.13	.07	--
2.35	.19	.15	.22	2.85	.12	.07	--
2.40	.17	.14	.20	2.90	.12	.06	--
2.45	.17	.13	.17	2.95	.12	--	--
2.50	.16	.12	.16	3.00	.11	--	--
				3.05	.11	--	--

The width bias test was performed for the simulated and observed hydrograph widths at 50 and 75 percent of peak flow for the hydrographs from the 19 gaged streams used in the verification test. It is assumed that, on the average, the predicted width will equal the actual width. In practice, the average predicted difference for any set of data is somewhat larger or smaller than zero. Given the magnitude of the error and the number of observations in the data set, the student's *t*-test, a test involving confidence limits for the random variable *t* of the Student's *t*-distribution and used in testing hypotheses concerning the mean of a normal distribution with an unknown standard deviation, provides a statistical means of determining whether the mean difference between simulated and observed width deviates significantly from zero.

The average predictions for the 50- and 75-percent peak-flow widths were negative (underpredicted) and positive (overpredicted) in the AP region, both positive in the PD region, and both negative in the CP region. In all cases, the student's *t*-test indicated that the errors are not statistically significant at the 0.05 level of significance. This indicates that the widths of the simulated hydrographs used in the verification step are not biased.

Sensitivity to Peak Flow and Lagtime

Because the simulation technique requires that the dimensionless hydrographs be used in conjunction with peak flow and lagtime, it is valuable to know how errors in predicting these two variables will affect the width of the simulated hydrograph. The sensitivity of simulated

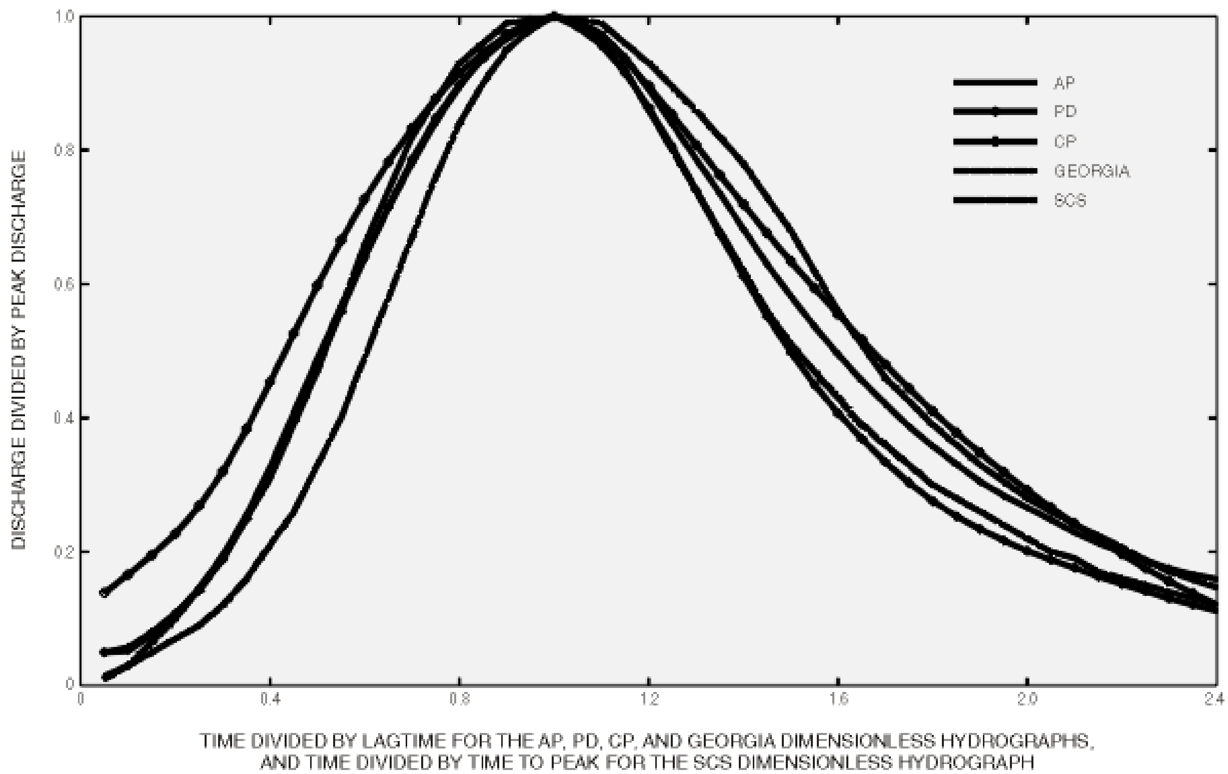

Figure 8. Dimensionless hydrographs for the Appalachian Plateaus and Allegheny Ridges (AP), Piedmont, Blue Ridge and Great Valley (PD), and Coastal Plain (CP), and the Georgia and U.S. Soil Conservation Service (SCS) dimensionless hydrographs with peaks aligned.

hydrograph widths to each of the two independent variables was determined by holding one variable constant while varying the value of the other, then determining the effect of the variation on the widths of the simulated hydrograph.

Following this procedure, when peak flow is varied, there is no change in the widths of the simulated hydrograph. When lagtime is varied, however, the simulated hydrograph widths are observed to vary directly by the same percentage. For example, if the estimate of lagtime is increased by 10 percent, the width of the simulated hydrograph will increase by 10 percent at all flow levels.

Hydrograph-Width Relations

In addition to allowing the simulation of peak-flow hydrographs, the information contained in the dimensionless hydrographs can be used to estimate the average length of time during which a particular discharge will be exceeded for a peak-flow event of a given magnitude. For example, if it is known that a facility or structure is inundated at or above a particular discharge, the time of inundation of the structure or facility can be estimated by using hydrograph-width relations derived from the dimensionless hydrographs.

Hydrograph-width ratios can be determined by subtracting the value of t/LT on the rising limb of a dimensionless hydrograph from the value of t/LT on the falling limb of the hydrograph corresponding to the same discharge ratio (Q/Q_p), over the full range of discharge ratios. The hydrograph-width ratios for each of the three dimensionless hydrographs are tabulated in table 7 and shown graphically in figure 9.

Table 7. *Relation of discharge ratios to hydrograph-width ratios for Maryland dimensionless hydrographs*

[*W*, hydrograph width; *LT*, basin lagtime; *Q*, discharge; Q_p, peak discharge]

Discharge ratio (Q/Q_p)	Width ratio (W/LT)		
	Appalachian Plateaus and Allegheny Ridges	Piedmont, Blue Ridge, and Great Valley	Coastal Plain
1.00	0.00	0.00	0.00
.95	.27	.25	.28
.90	.38	.37	.42
.85	.48	.46	.53
.80	.57	.54	.64
.75	.66	.62	.74
.70	.74	.70	.85
.65	.82	.76	.94
.60	.91	.83	1.04
.55	1.00	.91	1.14
.50	1.08	.99	1.24
.45	1.19	1.07	1.34
.40	1.29	1.16	1.45
.35	1.40	1.26	1.58
.30	1.54	1.36	1.70
.25	1.70	1.50	1.86
.20	1.90	1.69	2.04

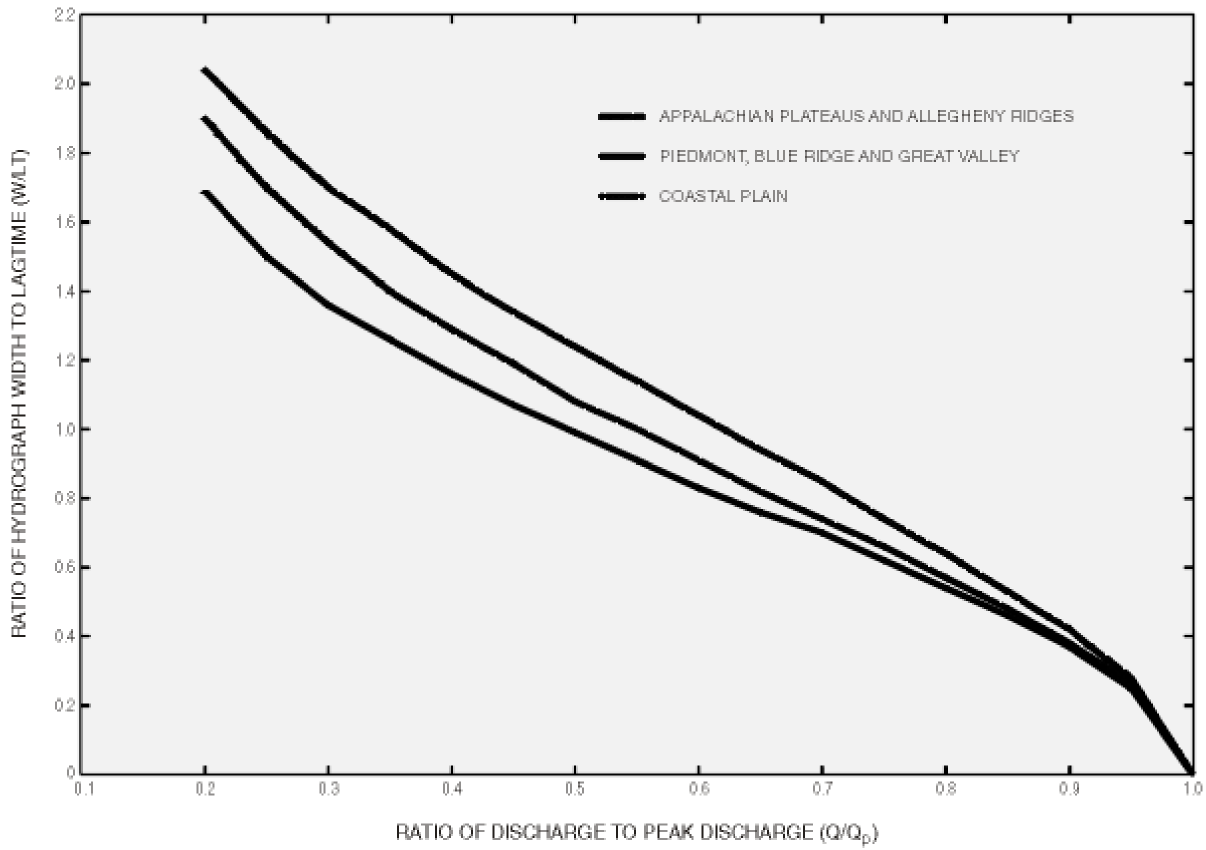

Figure 9. Hydrograph-width relations for Maryland dimensionless hydrographs.

The width (W) of a simulated hydrograph, in hours, corresponding to the average length of time a particular discharge (Q) will be exceeded, can be estimated by computing Q/Q_p, then multiplying the corresponding W/LT ratio from table 7 by the appropriate estimate of lagtime.

ADJUSTMENT FOR CORRECT RUNOFF VOLUME

During testing of the simulation technique, plots of the runoff-volume percent difference and peak discharge were made for all basins in the data set. The purpose of the plots was to check for bias in the runoff volumes of simulated hydrographs. The plots required actual peak discharges and runoff volumes for all rainfall-runoff events in the data base, and estimated runoff volumes from the dimensionless hydrographs corresponding to each rainfall-runoff event. Actual peak discharges and runoff volumes were available from the data base. Estimated runoff volumes were obtained by the following procedure explained in Bohman (1990).

The general expression for runoff volume for each of the dimensionless hydrographs is:

$$V_{DH} = (K)(Q_P)^{1.0}(LT_{RR})^{1.0}(A)^{-1.0} \quad , \qquad (3)$$

where

V_{DH} is dimensionless hydrograph runoff volume, in inches;

K is a conversion constant;

Q_p is peak discharge, in cubic feet per second;

LT_{RR} is lagtime of a particular rainfall-runoff event, in hours; and

A is as defined for equation 1.

In order to use equation 3 to estimate runoff volumes, the appropriate value of K must be calculated for each dimensionless hydrograph. This is done by extrapolating the rising and falling limbs of each dimensionless hydrograph to a discharge ratio of zero. The discharge-ratio ordinates are then summed at time-ratio intervals of 0.05. The sum is multiplied by conversion constants for time and length to make the equation dimensionally consistent and provide runoff volume in units of inches. The results of this procedure for the three dimensionless hydrographs used in this report are:

$K_{AP} = 0.00140$

$K_{PD} = 0.00162$

$K_{CP} = 0.00214$

Using the values of K along with equation 3, a runoff volume estimate was calculated for each rainfall-runoff event in the data base and volume-bias plots were made for each study region.

The plots showed that the technique was biased toward overprediction of the volumes of hydrographs in the PD and the CP study regions. Data from the AP study region showed no bias. A volume correction factor (*VCF*) was developed for each of the two affected study regions in order to correct the overprediction bias.

Applying a *VCF* to the basin lagtime estimated using equation 1 preserves the shape of the simulated hydrograph and provides a more accurate simulation. The *VCF* is defined as the ratio of the regression estimate of the actual runoff volume to the runoff volume from the simulated hydrograph. Based on data from 271 events at 71 stream-gaging stations, regression equations relating actual runoff volume to drainage area, peak discharge, and lagtime for the two regions showing volume-prediction bias are as follows:

$$V_{PD} = 0.00152 \; (Q_P)^{1.02}(LT_{RR})^{0.870}(A)^{-0.990} \quad , \qquad (4)$$

$$V_{CP} = 0.00336(Q_P)^{0.970}(LT_{RR})^{0.643}(A)^{-0.878} \quad , \qquad (5)$$

where

V_{PD} and V_{CP} are regression-derived runoff volumes, in inches, for the Piedmont, Blue Ridge, and Great Valley and Coastal Plain study regions, respectively; and

Q_p, LT_{RR}, and A are as defined in equation 3 earlier in this section.

The average standard errors of prediction for equations 4 and 5 are 39 (+46/-32) percent and 28 (+32/-24) percent, respectively.

Applying the definition of the *VCF*, and using equations 3, 4, and 5, the volume correction factors are as follows:

$$VCF_{PD} = 0.939(Q_P)^{0.020}(LT)^{-0.130}(A)^{0.010} \quad , \qquad (6)$$

$$VCF_{CP} = 1.568(Q_P)^{-0.030}(LT)^{-0.357}(A)^{0.122} \quad , \qquad (7)$$

where

VCF_{PD} and VCF_{CP} are the correction factors for the Piedmont, Blue Ridge, and Great Valley and Coastal Plain study regions, respectively; and

Q_p, LT, and A are as defined in equation 3 earlier in this section.

Applying the appropriate volume correction factor to the estimated basin lagtime (*LT*) from equation 1 for the PD and CP study regions removes the bias in the estimates of hydrograph volume for those regions. No volume correction is required for the AP study region.

VERIFICATION OF THE SIMULATION TECHNIQUE

By definition, peak-flow events characterized as having recurrence intervals of 100 years or greater are relatively rare events. Almost all of the data used to develop the simulation technique correspond to peak flows with recurrence intervals of less than 100 years. It is anticipated, however, that this technique will often be used to simulate hydrographs for 100-year events or larger. So that the user has a measure of accuracy for the results of the technique when applied to extreme events, an independent set of data containing the hydrographs of the peaks-of-record at 20 stream-gaging stations in the data base was assembled for verification of the entire simulation technique.

The 20 drainage basins for which peak-of-record hydrograph data were available had drainage areas ranging between 1.91 and 281 mi^2. The recurrence intervals of the selected peak flows range from 45 to 400 years, with an average recurrence interval of 130 years. After determining the recurrence interval for each peak flow, the appropriate regional peak-flow estimation equations from Dillow (1996) were used to compute the corresponding peak discharges (Q_p) to be used for hydrograph simulation, interpolating where necessary. The appropriate lagtime estimate was calculated for each event using equation 1, in conjunction with equations 6 and 7 where appropriate.

These estimates of peak discharge and lagtime were then used to simulate peak-flow hydrographs. The average standard errors of prediction obtained from comparing the 50 and 75 percent of peak-flow widths of the simulated hydrographs to those of the observed hydrographs are +/- 61 percent and +/- 56 percent, respectively. Table 8 lists the estimated and observed hydrograph widths used to compute these errors. Figure 10 illustrates an

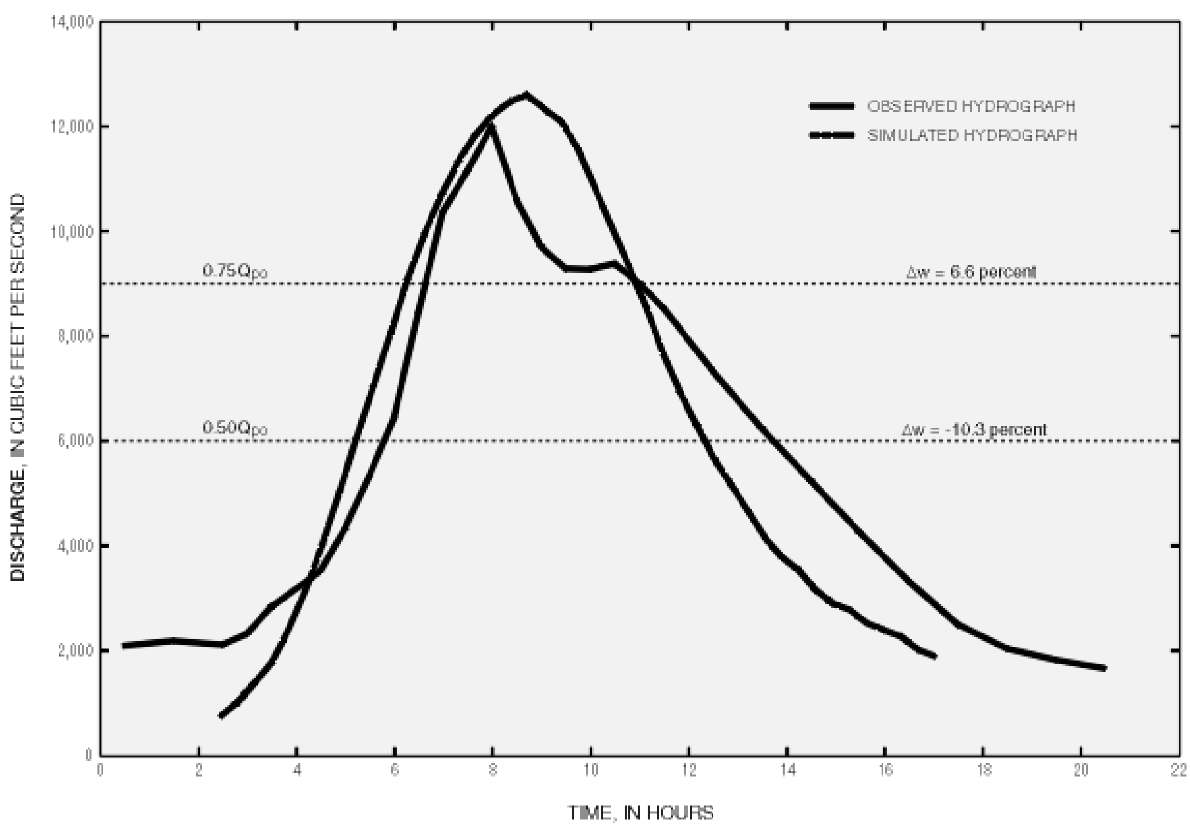

Figure 10. Observed and simulated hydrographs for width comparison at 50 and 75 percent of peak flow for Catoctin Creek near Middletown, Maryland (station no. 01637500), October 9, 1976.

Table 8. Observed and estimated hydrograph widths and peak flows for 20 peaks of record

[hrs, hours; ft^3/s, cubic feet per second; Q_{po}, observed peak discharge;----, data undefined]

Station no.	Observed width (hrs)		Estimated width (hrs)		Peak flow (ft^3/s)	
	$0.50Q_{po}$	$0.75Q_{po}$	$0.50Q_{po}$	$0.75Q_{po}$	Observed (Q_{po})	Estimated
01483200	12.10	7.12	9.65	7.27	712	999
01483700	28.08	15.29	17.11	11.36	1,900	2,110
01484000	13.75	6.75	9.05	----	2,090	1,550
01484500	15.40	7.63	14.56	11.55	303	521
01493500	8.52	5.45	11.72	8.17	7,500	8,940
01582000	13.33	8.23	7.47	5.00	8,280	9,030
01585100	3.95	1.90	2.64	1.18	8,000	6,720
01585500	1.92	1.05	3.21	2.06	2,220	2,250
01590500	6.17	3.33	6.34	----	2,100	1,540
01591000	2.58	1.52	6.05	3.23	21,800	19,500
01593500	7.13	4.33	6.22	3.64	12,400	11,700
01594936	2.37	1.48	3.12	----	895	552
01596500	6.92	3.67	15.23	9.42	7,510	7,670
01619500	6.92	4.00	13.96	9.17	12,600	13,300
01637500	7.93	4.37	7.11	4.66	12,000	12,600
01639500	8.13	3.80	9.21	4.95	28,000	25,100
01649500	10.40	2.53	7.17	4.94	10,600	12,200
01651000	1.25	.77	5.73	3.09	18,000	16,200
01661500	4.70	2.73	12.49	6.05	7,950	7,050
03078000	11.42	7.33	19.73	12.13	8,400	8,530

example of the comparison of a simulated hydrograph with an observed hydrograph.

LIMITATIONS OF TECHNIQUE

Use of the hydrograph simulation technique described herein should be limited to drainage basins in Maryland with drainage areas, main channel slopes, forest cover, and impervious area within the ranges shown by the data in table 3. The ranges are: drainage area, 0.15 to 494 mi^2; main channel slope, 1.49 to 1,000 ft/mi; forest cover, 2 to 100 percent; and impervious area, 0 to 40.8 percent. The expected errors for drainage basins with characteristics outside these ranges are unknown, and may not be assumed to be comparable with the expected errors provided for the technique. The technique is not valid for use in simulating snowmelt runoff hydrographs, or directly simulating complex, multi-peaked hydrographs. Also, the technique is not valid for use on drainage basins with regulated peak flow, unless estimates of peak discharge and lagtime are available that accurately account for the effects of regulation.

The technique provides an estimate of the average hydrograph associated with the peak discharge of a given recurrence interval for a drainage basin. When used for design purposes, results will be accurate within the limits stated in this report. The technique is not intended to be used to estimate hydrographs for comparison with recorded hydrographs.

HYDROGRAPH SIMULATION TECHNIQUE

In order to obtain results with accuracy within the limits stated in this report, the most current or best available maps and (or) computer data bases, such as U.S. Geological Survey topographic quadrangle maps or land-use data from the Maryland Office of State Planning (1991), should be used in carrying out hydrograph simulations. The technique for simulating a peak-flow hydrograph is defined by the following steps.

(1) Delineate and measure the drainage area associated with the site on the best available topographic maps. Using figure 2, or the best available Fall Line delineation, determine the study region(s) in which the basin is located. Because the Piedmont, Blue Ridge, and Great Valley (PD) and Coastal Plain (CP) regions are hydrologically connected, it is possible that the basin characterized by the site is in both regions. If this is the case, determine the percentages of the drainage area in each region. The Appalachian Plateaus and Allegheny Ridges (AP) region is hydrologically independent and does not share drainage basins with any other study region.

(2) Compute the peak discharge (Q_p) of interest using current peak-flow estimation techniques. Dillow (1996) and Sauer and others (1983) are examples of some appropriate methods.

(3) Compute the basin lagtime (LT) for the site using equation 1. Use of equation 1 requires knowledge of the location of the drainage basin with respect to the study regions defined in the report, and values of drainage area (A), main channel slope (SL), forest cover (F), and impervious area (IA).

The region(s) in which the basin is located is determined in step 1 above. If the drainage basin lies partially in both the PD and CP study regions, compute two values of LT, one for each region, as if the basin was contained entirely within that region. Use the drainage area measured in step 1 and compute the main channel slope as defined in the Glossary. Measure the percentages of forest cover and other land-use types existing in the drainage basin. Using the average percentages of impervious area associated with each standard land use from the U.S. Department of Agriculture (1975), compute the percentage of impervious area exhibited by the basin.

(4) If the drainage basin occurs in the AP region, proceed to step 6. If the drainage basin occurs in either the PD or CP regions, then the appropriate volume correction factor (*VCF*), calculated by using equation 6 or 7, must be applied to the estimated basin lagtime to produce a simulated hydrograph with appropriate volume. If the *VCF* is not applied in these regions, the typical simulated hydrograph will overestimate the total volume of flow for a given event. If the drainage basin lies partially in both the PD and CP study regions, compute two values of *VCF* for the drainage basin, one for each region.

(5) Apply each *VCF* to the corresponding *LT*. If the drainage basin is entirely within either the PD or CP study region, multiply the *LT* by the *VCF* to obtain the volume-corrected basin lagtime (*VLT*), then proceed to step 6. Otherwise, calculate an average value of *VLT* by multiplying both values of *VLT* by the corresponding percentages of the drainage basin contained in the PD and CP regions. Add these two quantities to find the weighted average value of *VLT* for the drainage basin. Note that the *VLT* is a quantity to be used in simulating a peak-flow hydrograph by methods discussed in this report, and should not be confused or equated with basin lagtime.

(6) Select the dimensionless hydrograph for the region containing the drainage basin from table 6.

If the basin spans both the PD and CP regions, calculate a set of ordinates (Q/Q_p) by weighting the ordinates of the dimensionless hydrographs for the PD and CP regions, using the percentages of the drainage area within each region as the weighting factors, and summing the resulting values for each time ratio increment.

(7) Using the values of Q_p and the appropriate lagtime value, either *LT* in the AP region or *VLT* in the PD and CP regions, along with the coordinates of the selected dimensionless hydrograph, the peak-flow hydrograph can be simulated. Calculate the coordinates of the simulated hydrograph by multiplying the values of *t/LT* from table 6 by either *LT* or *VLT* to obtain the values for time, and multiply the values of Q/Q_p selected or calculated from table 6 by Q_p to obtain the corresponding values of discharge.

SIMULATING A PEAK-FLOW HYDROGRAPH

The following example illustrates the use of the hydrograph simulation technique. The site chosen for the example is at U.S. Geological Survey stream-gaging station 01649500 on Northeast Branch Anacostia River at Riverdale, Md. The example will simulate the average hydrograph associated with the 100-year peak discharge by following steps 1 through 7 as explained in the previous section.

***Step 1**:* After delineating the basin on the best available topographic maps, comparing the maps with figure 2 indicates that the drainage basin lies in both the CP and PD study regions. Using the delineation on the best available topographic map, and measuring the parts of the basin in each region, it is determined that 20 percent of the basin lies in the PD study region. The remaining 80 percent is in the CP study region.

Estimating Peak Flow

***Step 2**:* The peak discharge of the 100-year recurrence interval is calculated using the methods described in Dillow (1996) and Sauer and others (1983), although any documented peak-flow estimation method can be used for this procedure. In this example, drainage area (*A*), forest cover (*F*), and basin development factor (*BDF*) are required to estimate the peak discharge.

Dillow (1996) requires drainage area and forest cover to estimate peak flow in both the Piedmont and in the Western Coastal Plain regions defined in that report. The applicable equations for peak-flow estimation are as follows:

$$Q_{100} = 3,060A^{0.557}(F+10)^{-0.241} \quad \textit{(Piedmont region)}, \qquad (8)$$

$$Q_{100} = 2,140A^{0.770}(F+10)^{-0.391} \quad \textit{(Western Coastal Plain region)}, \qquad (9)$$

where

A is the drainage area, in square miles;

F is the forest cover, in percent.

Using the basin delineation on the best available topographic maps, the drainage area is measured to be 72.8 mi^2. Using the most current available land-use data, in this case from the Maryland Office of State Planning (1991), 33 percent of the basin area is characterized by forest cover. Applying these values to equations 8 and 9 gives the following results:

$$Q_{100} = 3,060(72.8)^{0.557}(33+10)^{-0.241} = 13,467 \text{ ft}^3\text{/s} \quad \textit{(Piedmont region)}$$

$$Q_{100} = 2,140(72.8)^{0.770}(33+10)^{-0.391} = 13,353 \text{ ft}^3\text{/s} \quad \textit{(Western Coastal Plain region)}$$

The peak-flow estimation technique also requires that any estimate for a gaged stream site be weighted with flow estimates based on the observed data from the stream-gaging station using the following equation:

$$Q_w = [Q_g N_g + Q_r N_r]/(N_g + N_r) \quad , \qquad (10)$$

where

Q_w is the log of the weighted peak-flow estimate at the gaged location;

Q_g is the log of the discharge at the gaged location for the selected recurrence interval, derived from observed streamflow-gage data through the current year;

Q_r is the log of the discharge computed using the appropriate estimation equation from Dillow (1996) for the selected recurrence interval;

N_g is the number of years of record associated with the gaged location; and

N_r is the number of equivalent years of record for the selected estimation equation from Dillow (1996).

Using equation 10, peak-flow data available from the U.S. Geological Survey, and data available in Dillow (1996), with Q_g equal to 13,200 ft^3/s, N_g equal to 50 years, Q_r and N_r equal to 13,467 ft^3/s and 19 years, respectively, for the PD region, and Q_r and N_r equal to 13,353 ft^3/s and 13 years, respectively, for the CP region, the following weighted 100-year peak-flow estimates are obtained for each of the two regions:

Q_w = [log (13,200) (50) + log (13,467) (19)] / (50+19) = 4.1230, gives $10^{4.1230}$ = 13,273 ft^3/s, or approximately 13,300 ft^3/s for the Piedmont; and

Q_w = [log (13,200) (50) + log (13,353) (13)] / (50+13) = 4.1216, gives $10^{4.1216}$ =13,231 ft^3/s, or approximately 13,200 ft^3/s for the Western Coastal Plain.

A weighted average of these two values is derived according to the relative area of the basin in each region:

Q_{100} = (13,300 ft^3/s x 20 percent) + (13,200 ft^3/s x 80 percent)
= 2,660 ft^3/s + 10,560 ft^3/s
= 13,220 ft^3/s.

In accordance with U.S. Geological Survey policy regarding streamflow-measurement accuracy, this value is rounded to three significant figures, resulting in an estimate of the 100-year peak flow for a nondeveloped drainage basin of 13,200 ft ^{3}/s.

The technique used to calculate this estimate is only valid for nondeveloped drainage basins. According to the land-use data available from the Maryland Office of State Planning (1991) and the average percentages of impervious area from the U.S. Department of Agriculture (1975) that are characteristic of each land-use category, this basin is characterized as having 22 percent impervious area. In accordance with the criterion for identifying developed basins described in Sauer and others (1983), which states that any basin with residential, commercial, and industrial land-use areas that combine to comprise more than 15 percent of the total basin area is considered to be developed, this basin is considered to be developed. To account for the effects of development and obtain an accurate estimate of the 100-year peak flow, the technique described by Sauer and others (1983) was selected.

In addition to drainage area and the peak discharge estimate for an equivalent non-developed drainage basin, the technique of Sauer and others (1983) requires a value for basin development factor (*BDF*), an index of the prevalence of drainage aspects of (a) storm sewers, (b) channel improvements, (c) impervious channel linings, and (d) curb-and-gutter streets. The range of *BDF* is 0 to 12. A value of zero for *BDF* indicates the drainage aspects mentioned previously are not prevalent, but does not necessarily mean the basin is completely undeveloped. A value of 12 indicates full development of the drainage aspects throughout the basin. Refer to Sauer and others (1983) for a complete description of the procedure used to evaluate *BDF* for a basin. From field inspection, *BDF* in this example is determined to equal 5.

From Sauer and others (1983), an equation for estimating the 100-year peak flow in a developed drainage basin is as follows:

$$UQ100 = 7.70A^{0.15}(13 - BDF)^{-0.32}RQ100^{0.82} \quad , (11)$$

where

UQ100 is the 100-year peak discharge for a developed watershed, in cubic feet per second;

A is as defined in equation 8 earlier in this section;

BDF is the basin development factor as previously described in this section; and

RQ100 is the 100-year peak discharge for an equivalent nondeveloped drainage basin, in cubic feet per second.

Using equation 11 in conjunction with the previously measured drainage area, the value of *BDF* acquired through field inspection, and using the Q_{100} value calculated using equation 10 as the estimate of *RQ100*, the 100-year peak discharge, gives the estimate of 100-year discharge for the basin as follows:

$$UQ100 = 7.70\ (72.8)^{0.15}(13-5)^{-0.32}(13{,}200)^{0.82} = 18{,}017\ ft^3/s,$$
$$\text{or approximately } 18{,}000\ ft^3/s.$$

The estimate of the 100-year recurrence level Q_p for the basin in this example is 18,000 ft ^{3}/s.

Estimating Lagtime

Step 3: Since the drainage basin in this example is partially within both the PD and CP study regions, two values of LT will be calculated using equation 1,

$$LT = 0.18A^{0.234}\,SL^{-0.312}(101\text{-}F)^{-0.220}\,(101\text{-}IA)^{1.06}(10^{\,[0.219AP\,+\,0.202CP]}),$$

where

- LT is the basin lagtime, in hours;
- A is the drainage area, in square miles;
- SL is the main channel slope, in feet per mile;
- F is the forest cover, in percent;
- IA is the impervious area of the basin, in percent; and
- AP, CP are qualitative variables with discrete values of 0 or 1. A value of 1 is assigned when the basin for which lagtime is being estimated is in the corresponding study region, as defined in figure 2.

In addition to the data already listed for this example, the estimation of lagtime requires main channel slope (SL) in units of feet per mile, measured from the best available topographic maps.

The main channel slope is obtained as follows. Using the best available topographic maps, with the drainage divide for the basin drawn on, identify the main channel of the basin. On U.S. Geological Survey topographic maps, this can be done by following the blue line of the stream channel from the point of interest upstream, following the branch at each confluence which drains the larger upstream basin area. Extend the blue line from its uppermost end to the drainage divide following the natural drainage channel as shown by the topographic contours. Measure the total length of the channel from the point of interest to the drainage divide. Find the points along the main channel that are located at 10 and 85 percent of the main channel length as measured upstream from the point of interest. Using the map contours, estimate the elevations of these two points. To find the main channel slope, subtract the elevation of the upstream point from that of the downstream point and divide the difference by the quantity 0.75 times the total length of the main channel.

The basin lagtime (LT) must be calculated for both study regions using equation 1 as follows:

$$LT = 0.18A^{\,0.234}SL^{\,-0.312}(101\text{-}F)^{\,-0.220}(101\text{-}IA)^{1.06}(10^{\,[0.219AP\,+0.202CP]}),$$

by substitution,

$$LT_{PD} = 0.18\,(72.8)^{\,0.234}(27.2)^{\,-0.312}\,(101\text{-}33)^{\,-0.220}\,(101\text{-}22)^{\,1.06}\,(10^{\,[0.219\,(0)\,+\,0.202\,(0)]}) = 7.11 \text{ hours},$$

and

$$LT_{CP} = 0.18(72.8)^{\,0.234}\,(27.2)^{\,-0.312}(101\text{-}33)^{\,-0.220}\,(101\text{-}22)^{\,1.06}\,(10^{\,[0.219\,(0)\,+0.202\,(1)]}) = 11.32 \text{ hours}.$$

Step 4: Since the PD and CP study regions are involved, VCF values must also be calculated. Using equations 6 and 7,

$$VCF_{PD} = 0.939\,(Q_p)^{\,0.020}(LT)^{\,-0.130}(A)^{\,0.010},$$

$$VCF_{CP} = 1.568\,(Q_p)^{\,-0.030}(LT)^{\,-0.357}(A)^{\,0.122},$$

where

- Q_p is peak discharge, in cubic feet per second;
- LT is basin lagtime, in hours; and
- A is the drainage area, in square miles,

with the previously determined values of drainage area (A), main channel slope (SL), forest cover (F) data from the Maryland Office of State Planning (1991), impervious area (IA) calculated by using land-use data from the Maryland Office of State Planning (1991) and average impervious area values for various land-use types from the U.S. Department of Agriculture (1975), and 100-year peak discharge (Q_p), the user obtains the following two values of VCF:

$VCF_{PD} = 0.939(Q_p)^{0.020} (LT)^{-0.130} (A)^{0.010}$,

by substitution,

$VCF_{PD} = 0.939(18{,}000)^{0.020} (7.11)^{-0.130} (72.8)^{0.010} = 0.924$,

and

$VCF_{CP} = 1.568(Q_p)^{-0.030}(LT)^{-0.357}(A)^{0.122}$,

by substitution,

$VCF_{CP} = 1.568(18{,}000)^{-0.030}(11.32)^{-0.357}(72.8)^{0.122} = 0.829$.

Step 5: Using the values of *LT* and *VCF* calculated in steps 3 and 4, respectively, calculate values of *VLT* for each region as follows:

$VLT_{PD} = LT_{PD} \times VCF_{PD} =$ 7.11 hours $\times$ 0.924 = 6.57 hours,

$VLT_{CP} = LT_{CP} \times VCF_{CP} =$ 11.32 hours $\times$ 0.829 = 9.38 hours.

These estimates of *VLT* are weighted according to the relative area of the basin in each of the two regions to produce the *VLT* for the basin:

$VLT = (VLT_{PD}$ x 20 percent) + $(VLT_{CP}$ x 80 percent)
= (6.57 hours x 0.20) + (9.38 hours x 0.80)
= 1.31 hours + 7.50 hours
= 8.81 hours, or approximately 8.8 hours.

Expanding the Dimensionless Hydrograph

Step 6: The basin in this example lies partially in both the PD and CP study regions, so the ordinates for the dimensionless hydrograph are a weighted average of the PD and CP dimensionless hydrographs listed in table 6. Each ordinate for this example is the sum of 20 percent times the PD ordinate plus 80 percent times the CP ordinate, reflecting the percentages of the drainage basin area in each study region. For example, in this problem the weighted value of the ordinate corresponding to the 0.05 time ratio would be obtained as follows. Since the ordinates corresponding to that time ratio are 0.00 and 0.06 for the PD and CP dimensionless hydrographs, respectively, the weighted ordinate value is

[(0.2) x (0.00)] +[(0.8) x(0.06)] = 0.048 or approximately 0.05.

Step 7: The weighted ordinates and their corresponding time ratios can be expanded to simulate the peak-flow hydrograph. Using

Q_p =18,000 ft^3/s and VLT = 8.8 hours,

the weighted average PD/CP dimensionless hydrograph can be expanded as shown in table 9 to produce the simulated hydrograph seen in figure 11.

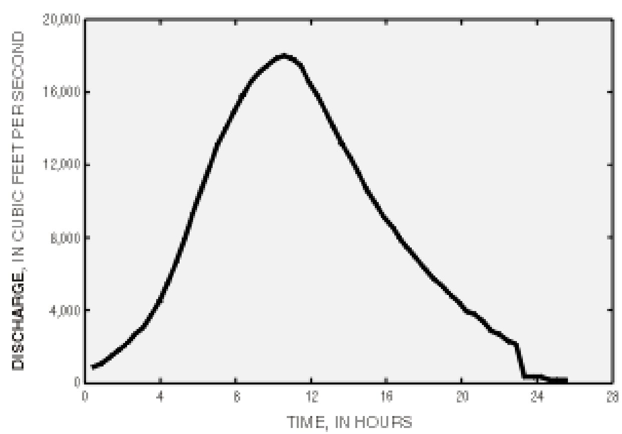

Figure 11. Simulated 100-year peak-flow hydrograph for Northeast Branch Anacostia River at Riverdale, Maryland.

Table 9. *Simulated coordinates of the 100-year peak-flow hydrograph for Northeast Branch Anacostia River at Riverdale, Maryland*

[*t*, time; *LT*, basin lagtime; *VLT*, volume-corrected basin lagtime; hrs, hours; *Q*,discharge; *Qp*, peak discharge; ft^3/s, cubic feet per second; col., column]

Time ratio (*t/LT*) (from table 6)	Volume-corrected basin lagtime (*VLT*) (hours)	Time (hrs) (col. 1 x col. 2)	Discharge ratio (Q/Q_p) (from table 6)	Peak discharge (Q_p) (ft^3/s)	Discharge (*Q*) (ft^3/s) (col. 4 x col. 5)
0.05	8.8	0.44	0.05	18,000	900
.10	8.8	.88	.06	18,000	1,080
.15	8.8	1.32	.08	18,000	1,440
.20	8.8	1.76	.10	18,000	1,800
.25	8.8	2.20	.12	18,000	2,160
.30	8.8	2.64	.15	18,000	2,700
.35	8.8	3.08	.17	18,000	3,060
.40	8.8	3.52	.21	18,000	3,780
.45	8.8	3.96	.25	18,000	4,500
.50	8.8	4.40	.31	18,000	5,580
.55	8.8	4.84	.37	18,000	6,660
.60	8.8	5.28	.44	18,000	7,920
.65	8.8	5.72	.52	18,000	9,360
.70	8.8	6.16	.59	18,000	10,620
.75	8.8	6.60	.66	18,000	11,880
.80	8.8	7.04	.73	18,000	13,140
.85	8.8	7.48	.78	18,000	14,040
.90	8.8	7.92	.83	18,000	14,940
.95	8.8	8.36	.88	18,000	15,840
1.00	8.8	8.80	.92	18,000	16,560
1.05	8.8	9.24	.95	18,000	17,100
1.10	8.8	9.68	.97	18,000	17,460
1.15	8.8	10.12	.99	18,000	17,820
1.20	8.8	10.56	1.00	18,000	18,000
1.25	8.8	11.00	.98	18,000	17,820
1.30	8.8	11.44	.96	18,000	17,460

Table 9. *Simulated coordinates of the 100-year peak-flow hydrograph for Northeast Branch Anacostia River at Riverdale, Maryland--Continued*

Time ratio (*t/LT*) (from table 6)	**Volume-corrected basin lagtime (*VLT*) (hours)**	**Time (hrs) (col. 1 x col. 2)**	**Discharge ratio (Q/Q_p) (from table 6)**	**Peak discharge (Q_p) (ft^3/s)**	**Discharge (Q) (ft^3/s) (col. 4 x col. 5)**
1.35	8.8	11.88	0.92	18,000	16,560
1.40	8.8	12.32	.88	18,000	15,840
1.45	8.8	12.76	.83	18,000	14,940
1.50	8.8	13.20	.78	18,000	14,040
1.55	8.8	13.64	.73	18,000	13,140
1.60	8.8	14.08	.69	18,000	12,420
1.65	8.8	14.52	.64	18,000	11,520
1.70	8.8	14.96	.59	18,000	10,620
1.75	8.8	15.40	.55	18,000	9,900
1.80	8.8	15.84	.51	18,000	9,180
1.85	8.8	16.28	.48	18,000	8,640
1.90	8.8	16.72	.44	18,000	7,920
1.95	8.8	17.16	.41	18,000	7,380
2.00	8.8	17.60	.38	18,000	6,840
2.05	8.8	18.04	.35	18,000	6,300
2.10	8.8	18.48	.32	18,000	5,760
2.15	8.8	18.92	.30	18,000	5,400
2.20	8.8	19.36	.27	18,000	4,860
2.25	8.8	19.80	.25	18,000	4,500
2.30	8.8	20.24	.22	18,000	3,960
2.35	8.8	20.68	.21	18,000	3,780
2.40	8.8	21.12	.19	18,000	3,420
2.45	8.8	21.56	.16	18,000	2,880
2.50	8.8	22.00	.15	18,000	2,700
2.55	8.8	22.44	.13	18,000	2,340
2.60	8.8	22.88	.12	18,000	2,160
2.65	8.8	23.32	.02	18,000	360

Table 9. *Simulated coordinates of the 100-year peak-flow hydrograph for Northeast Branch Anacostia River at Riverdale, Maryland--Continued*

Time ratio (*t/LT*) (from table 6)	Volume-corrected basin lagtime (*VLT*) (hours)	Time (hrs) (col. 1 x col. 2)	Discharge ratio (Q/Q_p) (from table 6)	Peak discharge (Q_p) (ft^3/s)	Discharge (*Q*) (ft^3/s) (col. 4 x col. 5)
2.70	8.8	23.76	0.02	18,000	360
2.75	8.8	24.20	.02	18,000	360
2.80	8.8	24.64	.01	18,000	180
2.85	8.8	25.08	.01	18,000	180
2.90	8.8	25.52	.01	18,000	180

SUMMARY AND CONCLUSIONS

Three dimensionless hydrographs, a lagtime estimation equation, and two volume correction factors were developed to simulate peak-flow hydrographs in Maryland. When combined with a peak-discharge estimate of specific recurrence interval, these products can be used to simulate a hydrograph for any nonregulated stream site in Maryland. Simulating a hydrograph requires that estimates of peak discharge and lagtime be applied to the time and discharge ratios of the appropriate dimensionless hydrograph.

The dimensionless hydrographs were developed using data from 205 rainfall-runoff events on 62 gaged streams in Maryland and Delaware. The shapes of the hydrographs were verified using data from 73 events on 19 gaged streams not used in hydrograph development.

The equation for estimating basin lagtime was developed using multiple regression analysis techniques. Data from 80 of the 81 gaged drainage basins in the data base were used to develop the equation. One basin was excluded from the analyses because accurate rainfall data were not available to calculate the observed basin lagtime. The equation requires drainage area, main channel slope, forest cover, and impervious area, along with two qualitative variables accounting for regional biases, to estimate basin lagtime. Two volume-correction equations were developed to remove a hydrograph-volume prediction bias found while testing the simulation technique.

The three dimensionless hydrographs, the basin lagtime estimation equation, and the two volume-correction equations were applied to an independent data set containing the peak-of-record runoff event for 20 gaged basins in the data base in order to verify the accuracy of the simulation technique when applied to extreme events. The 20 observed hydrographs were not used in developing the technique, and had an average recurrence interval of 130 years. The results of comparing the observed and simulated hydrograph widths at the 50 and 75 percent of observed peak-flow levels were average standard errors of prediction of +/- 61 percent and +/- 56 percent, respectively. Accuracy of the simulation results within the stated average standard errors of prediction is ensured by adherence to the methods defining the technique and the guidelines set forth in the Limitations of Technique section.

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