ESTIMATING FLOOD HYDROGRAPHS FOR URBAN BASINS IN NORTH CAROLINA

By Robert R. Mason, Jr. *and* Jerad D. Bales

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CONVERSION FACTORS

Multiply	By	To obtain
	Length	
inch (in.)	25.4	millimeter
foot (ft)	0.3048	meter
foot per mile (ft/mi)	0.1894	meter per kilometer
mile (mi)	1.609	kilometer
square mile (mi^2)	2.590	square kilometer
	Area	
square foot (ft^2)	0.09290	square meter
	Flow	
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

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ABSTRACT

A dimensionless hydrograph for North Carolina was developed from data collected in 29 urban and urbanizing basins in the State. The dimensionless hydrograph can be used with an estimate of peak flow and basin lagtime to synthesize a design flood hydrograph for urban basins in North Carolina. Peak flows can be estimated from a number of available techniques; a procedure for estimating basin lagtime from main channel length, stream slope, and percentage of impervious area was developed from data collected at 50 sites and is presented in this report. The North Carolina dimensionless hydrograph provides satisfactory predictions of flood hydrographs in all regions of the State except for basins in or near Asheville where the method overestimated 11 of 12 measured hydrographs. A previously developed dimensionless hydrograph for urban basins in the Piedmont and upper Coastal Plain of South Carolina provides better flood-hydrograph predictions for the Asheville basins and has a standard error of 21 percent as compared to 41 percent for the North Carolina dimensionless hydrograph.

INTRODUCTION

From 1988 to 1993, a study was conducted to measure runoff characteristics of small urban basins in North Carolina, and to develop techniques for estimating peak flows (Robbins and Pope, 1996) and flood hydrographs for ungaged urban basins in the State. The study, which was conducted in cooperation with the North Carolina Department of Transportation and the Cities of Asheville, Fayetteville, and Raleigh, North Carolina, included measurements of streamflow and precipitation at 24 sites and compilation of existing data from 31 additional sites (fig. 1; table 1). During the first phase of the study, relations were developed for predicting peak flows in ungaged urban basins in North Carolina. Peak flows having recurrence intervals of 2-, 5-, 10-, 25-, 50-, and 100-years can be predicted from basin drainage area, impervious area, and the rural equivalent peak discharge (Robbins, and Pope, 1996). The relations generally have prediction errors of less than 40 percent.

In addition to peak discharges, knowledge of the time distribution of runoff is useful in the design of flood-control structures, stormwater drainage structures, culverts, and bridges. In selecting hydraulic designs, engineers attempt to maximize flood protection while minimizing total costs. Information on the complete flood hydrograph is required to optimize hydraulic designs and to evaluate risks associated with flood inundation. This information is particularly critical for urban areas where risks to lives and property from flooding are often greatest. Methods for formulating flood hydrographs for ungaged urban basins were developed in the second phase of the urban runoff study and are the subject of this report.

A flood hydrograph represents the time distribution of runoff in response to that portion of storm rainfall which is in excess of infiltration. Certain general characteristics are often used to analyze a flood hydrograph or to synthesize a hydrograph for design purposes. These characteristics include peak flow (Q_p), basin lagtime (L_t), and storm duration (fig. 2).

Flood hydrographs have been recorded at numerous rural sites in North Carolina. However, for ungaged basins, the flood hydrograph must be estimated, or synthesized, using any one of several available techniques, including application of a dimensionless hydrograph.

Purpose and Scope

This report presents a technique for estimating flood hydrographs for urban basins in North Carolina and documents the development of this method. The method is based on the application of a dimensionless hydrograph described in this report. Flood hydrographs can be synthesized by using the dimensionless

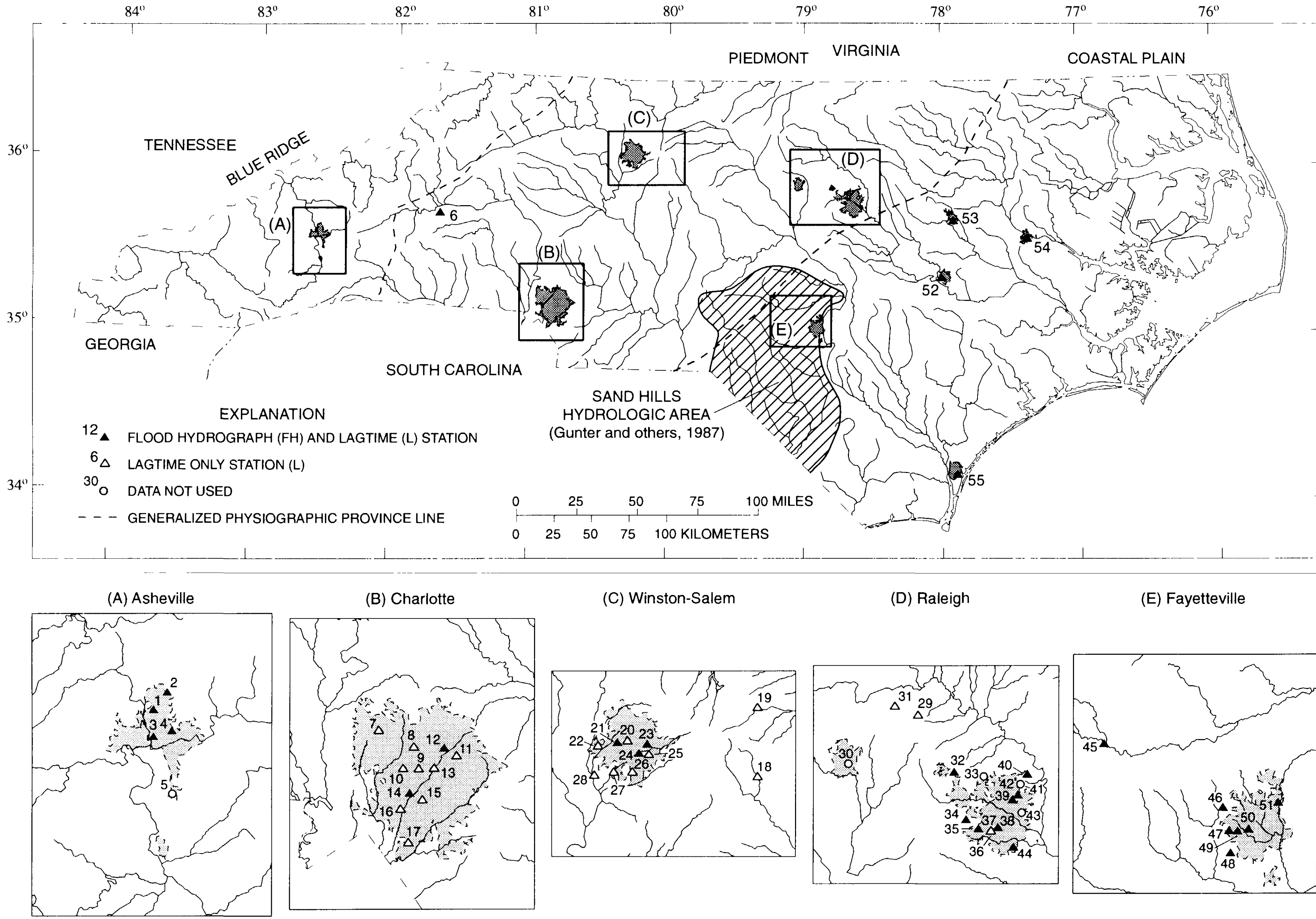

Figure 1. Locations of data-collection sites, North Carolina.

Table 1. Data-collection site descriptions in North Carolina

[Ave., avenue; FH/L, flood hydrograph and lagtime; L, lagtime only; nr., near; Rd., road; ---, data not used; trib., tributary; Blvd., Boulevard; SR, State Road]

Site number (fig. 1)	Station name	USGS station number	Latitude	Longitude	Use of data	Period of record
1	Reed Creek above Barnard Ave. at Asheville	03451510	35°36'52"	82°33'41"	FH/L	1986-88; 1989-92
2	Spooks Branch nr. Woodfin	0345153800	35°38'17"	82°32'24"	FH/L	1987-90
3	Nasty Branch at Asheville	0345112600	35°34'44"	82°33'35"	FH/L	1986-88
4	Ross Creek at Beaucatcher Rd. at Asheville	0345092550	35°35'15"	82°31'49"	FH/L	1986-89
[a]5	Dingle Creek nr. Skyland	03448068	35°30'22"	82°31'30"	---	---
[b]6	Hunting Creek at Morganton	02139610	35°44'17"	81°40'45"	FH/L	1967-70
[b]7	Paw Creek trib. 2 at Allenbrook Dr., Charlotte	02142950	35°15'40"	80°54'48"	L	1966-69
[b]8	Irwin Creek trib. at Charlotte	02146235	35°14'12"	80°50'50"	L	1966-69
[b]9	Little Sugar Creek at Brunswick Ave., Charlotte	02146409	35°12'11"	80°50'15"	L	1964-66
[b]10	Stewart Creek at Charlotte	02146280	35°12'11"	80°51'59"	L	1962-69
[b]11	Briar Creek trib. 6 at Sudbury Rd., Charlotte	02146435	35°13'27"	80°46'01"	L	1966-69
[b]12	Briar Creek trib. 7 at Shamrock Dr., Charlotte	02146436	35°14'07"	80°47'26"	FH/L	1966-70
[b]13	Briar Creek at East Seventh St., Charlotte	02146440	35°12'16"	80°48'33"	L	1962-69
[b]14	Little Hope Creek at Seneca Place, Charlotte	02146470	35°09'53"	80°51'12"	FH/L	1982-91
[b]15	Little Sugar Creek trib. 7 at Burnley Rd., Charlotte	02146505	35°09'19"	80°49'46"	L	1966-69
[b]16	McMullen Creek at Sharon View Rd. nr. Charlotte	02146700	35°08'27"	80°52'10"	L	1962-69
[b]17	McMullen Creek nr. Griffith	02146725	35°05'22"	80°51'16"	L	1962-69
[b]18	East Fork Deep River nr. High Point	02099000	36°02'15"	79°56'46"	L	1928-69
[b]19	Mill Creek nr. Stanleyville	02115730	36°10'49"	79°56'46"	L	1964-69
[b]20	Silas Creek at Winston-Salem	02115760	36°06'35"	80°16'19"	L	1964-69
[b]21	Silas Creek trib. at Pine Valley Road, Winston-Salem	02115765	36°06'19"	80°17'52"	FH/L	1968-70
[b]22	Brushy Creek at Winston-Salem	02115840	36°05'57"	80°20'46"	L	1964-69
[b]23	Brushy Creek trib. 2 at U.S. Hwy. 311, Winston-Salem	02115839	36°06'10"	80°13'21"	FH/L	1968-70
[b]24	Tar Branch at Walnut St., Winston-Salem	02115843	36°05'02"	80°14'34"	FH/L	1967-70
[b]25	Peters Creek at Winston-Salem	02115845	36°04'56"	80°13'04"	L	1964-69
[b]26	Fiddlers Creek nr. Winston-Salem	02115870	36°02'46"	80°15'30"	L	1964-69
[b]27	Silas Creek at Clemmons	02115800	36°02'44"	80°18'20"	L	1964-69
[b]28	Little Creek nr. Clemmons	02115810	36°02'19"	80°21'15"	L	1964-69
[b]29	Dye Creek at Guess Rd. (Durham)	02086760	36°01'09"	78°51'24"	L	1967-70
[a]30	Little Creek trib. nr. Chapel Hill	0209736050	35°55'02"	79°01'57"	---	---
[b]31	Third Fork Creek at University Dr. (Durham)	02097240	35°58'46"	78°54'54"	L	1964-69
32	Sycamore Creek nr. Lynn Crossroads	0208725600	35°54'03"	78°45'56"	FH/L	1987-91
[a]33	Hare Snipe Creek trib. nr. Leesville	0208726835	35°53'28"	78°41'25"	---	---
34	Richlands Creek nr. Westover	0208726100	35°48'13"	78°44'07"	FH/L	1987-93
35	Bushy Branch trib. below Schaub Dr. at Raleigh	0208734221 [c]0208734220	35°47'04"	78°42'14"	FH/L	1987-93
[b]36	Rocky Branch at Dan Allen Dr., Raleigh	02087349	35°46'55"	78°40'20"	L	1965-68
[b]37	Rocky Branch at Carmichael Gymnasium, Raleigh	02087350	35°46'50"	78°40'20"	L	1965-68
38	Pigeon House Creek at Cameron Village, Raleigh	0208732534	35°47'14"	78°39'17"	FH/L	1987-93
39	Big Branch trib. at Windgate Dr., Millbrook	0208730025	35°50'38"	78°37'01"	FH/L	1987-91
40	Perry Creek trib. at Neuse	0208721290	35°53'47"	78°34'46"	FH/L	1986-89
[a]41	Perry Creek at SR 2012, nr. Millbrook	0208721055	35°52'30"	78°35'48"	---	---
42	Marsh Creek at SR 2030, Millbrook	0208732810	35°51'13"	78°36'12"	FH/L	1986-89
[a]43	Marsh Creek nr. New Hope	0208732885	35°48'59"	78°35'37"	---	---
44	Walnut Creek trib. at Evers St., Raleigh	0208735550	35°44'49"	78°36'54"	FH/L	1987-90
45	Flat Creek nr. Inverness	02102908	35°10'54"	79°10'40"	FH/L	1991-92
46	Jack Fords Creek, Fayetteville	0210434115	35°05'35"	78°57'57"	FH/L	1989-92
47	Buckhead Creek at Skibo	0210438680	35°03'34"	78°57'17"	FH/L	1989-93
48	Buckhead Creek nr. Owens	02104387	35°01'37"	78°57'08"	FH/L	1976-80; 1989-92
49	Branson Creek nr. Fayetteville	0210397520	35°03'31"	78°56'23"	FH/L	1989-93
50	Hybart Creek trib. at Fayetteville	0210397475	35°03'41"	78°55'13"	FH/L	1989-92
51	Cape Fear River trib. nr. Fayetteville	0210367030	35°06'01"	78°52'03"	FH/L	1989-92
[d]52	Big Ditch at Retha St. at Goldsboro	02088682	35°22'16"	78°00'15"	FH/L	1980-84
[d]53	Hominy Swamp at Phillips St. at Wilson	02090512	35°42'39"	77°55'00"	FH/L	1978-85
[d]54	Greenmill Run at Arlington Blvd. at Greenville	02084070	35°35'57"	77°22'17"	FH/L	1980-85
[d]55	Hewletts Creek at SR 1102 nr. Wilmington	02093229	34°11'28"	77°53'32"	FH/L	1976-90

[a] Data were not used to formulate regional unit flood hydrograph or basin lagtime relation.
[b] From Putnam (1972).
[c] Refers to separately numbered raingage.
[d] From Gunter and others (1987).

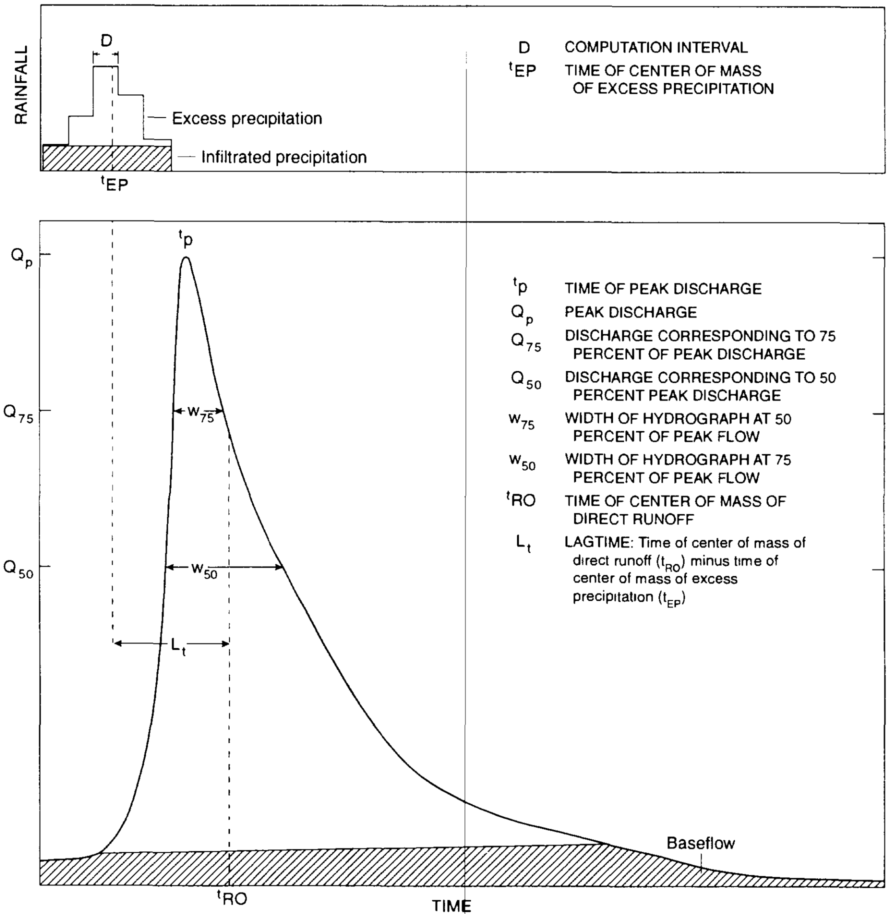

Figure 2. Generalized diagram showing hydrograph characteristics.

hydrograph and estimates of the hydrograph peak flow and basin lagtime. Estimates of peak flow can be determined by using the procedures of Robbins and Pope, (1996), or other appropriate methods. Methods for estimating basin lagtime also are presented in this report.

Available rainfall and runoff data from 29 urban basins in North Carolina, ranging in size from 0.04 to 9.10 square miles (mi^2), were used to develop the dimensionless hydrograph; lagtime data from 50 sites were used to develop the basin lagtime relation. Although the technique was developed by using data from urban basins, it may be applicable to rural basins as well. The technique is applicable only to basins without significant storage upstream from the point at which the flood hydrograph is estimated.

Acknowledgments

Special thanks are extended to several individuals who were instrumental in the establishment and support of this investigation, including Mr. Archie Hankins of the North Carolina Department of Transportation; Mr. James Ewing and Mr. Tom Tarrant of the City of Asheville; Mr. Robert Bennett and Mr. Gerald Croll of the City of Fayetteville; and Mr. Jimmie Beckom and Mr. Danny Bowden of the City of Raleigh.

Approach

The approach used in this study to develop the technique for estimating flood hydrographs is based on the unit hydrograph concept. The methods used to (1) determine unit hydrographs from measured streamflow and precipitation, (2) define the dimensionless hydrograph, and (3) estimate basin lagtime are described in this section of the report.

Unit Hydrographs

Generally, the shapes of flood hydrographs which result from storms of spatially uniform rainfall distributions are consistent for a given basin. The flood hydrographs that result from such storms usually retain the same overall shape but differ from storm to storm mainly in peak flow (primarily as a function of rainfall intensity) and width (primarily as a function of rainfall duration). This characteristic of a relatively consistent and linear response of runoff to uniform rainfall is the basis for the unit hydrograph concept.

The unit hydrograph is a hydrograph of direct runoff (total runoff less baseflow (fig. 2)) resulting from one unit (usually 1 inch) of excess rainfall uniformly generated over a basin at a uniform rate for a specified period of time or storm duration. The T-hour unit hydrograph is derived from a flood hydrograph produced by a storm of T-hours in duration by using equation 1, which is valid only if the excess rainfall is uniform (Chow, 1964; Viessman and others, 1977).

$$q(t) = \frac{(Q(t))}{EP} \tag{1}$$

where:

$q(t)$ is the ordinate of the T-hour unit hydrograph at time t;

$Q(t)$ is the ordinate of the T-hour flood hydrograph at time t; and

EP is the volume of excess precipitation.

However, most storms are composed of a series of nonuniform rainfall intensities of varying durations. Decomposing an observed flood hydrograph produced by nonuniform rainfall into the unit hydrograph can require complex matrix operations, statistical manipulations, or trial and error iterations (Snyder, 1955; Newton and Vineyard, 1967; Mays and Coles, 1980). Often, more than one solution is possible.

O'Donnell's (1960) method for deriving a unit hydrograph from a complex storm was used in this study. The method involves harmonic analysis of the rainfall excess and runoff data to derive a series of harmonic coefficients. Each increment of rainfall (having a duration equal to the data recording interval) is treated as an individual storm, and an instantaneous unit hydrograph that reproduces the direct runoff hydrograph is computed from the harmonic coefficients. For this study, direct runoff was computed by subtracting the baseflow (approximated as the flow beneath a line connecting the flows at the beginning and end of the hydrograph (fig. 2)), from the observed runoff. Rainfall excess was estimated as the storm rainfall minus a constant infiltration rate to account for the difference between total rainfall and total direct runoff (fig. 2).

Flood-hydrograph data (rainfall and streamflow records) from 34 sites in North Carolina (fig. 1; table 1) were reviewed to identify single-peaked flood hydrographs derived from storms of concentrated periods of rainfall. Computer programs developed by S.E. Ryan (U.S. Geological Survey, written commun., 1985) for application of the O'Donnell method were used to determine the unit hydrographs from the rainfall and streamflow data. A total of 101 unit hydrographs were developed. These hydrographs represent streams from many of the major urban centers of North Carolina, including Asheville (Blue Ridge Province), Charlotte, Raleigh, and Winston-Salem (Piedmont Province), Fayetteville (Sand Hills), and Wilson, Greenville, Goldsboro, and Wilmington (Coastal Plain Province).

Data for five study sites were not used in the hydrograph analysis. Hydrographs from Dingle Creek (site 5) were not used because most of the hydrographs at that site were multi-peaked, and as such, violate assumptions underlying the O'Donnell method. Data from Little Creek tributary near Chapel Hill (site 30) were not used because a satisfactory stage-discharge rating was not available. Likewise, hydrographs from Hare Snipe Creek tributary (site 33) were not used because a stage-discharge rating was not established for the site. Marsh Creek near New Hope (site 43) and Perry Creek at SR 2012 (site 41) were found to have significant detention storage upstream from the gage and, thus, were omitted from the analysis. Twenty-three of the remaining 29 flood-hydrograph basins were urban, and six were urbanizing.

A basin-average unit hydrograph for each of the remaining 29 flood-hydrograph basins was then developed by aligning the peaks of the storm unit hydrographs from each basin and averaging corresponding flow ordinates. The correct timing of the average flow ordinates was obtained by shifting the flow ordinates of the average hydrograph along the time axis until the centroid of the average hydrograph was aligned with the average of the centroids of the unit hydrographs for all of the storms for a basin (fig. 3). Resulting values on the time axis were rounded to the nearest whole interval of the computation period (table 2).

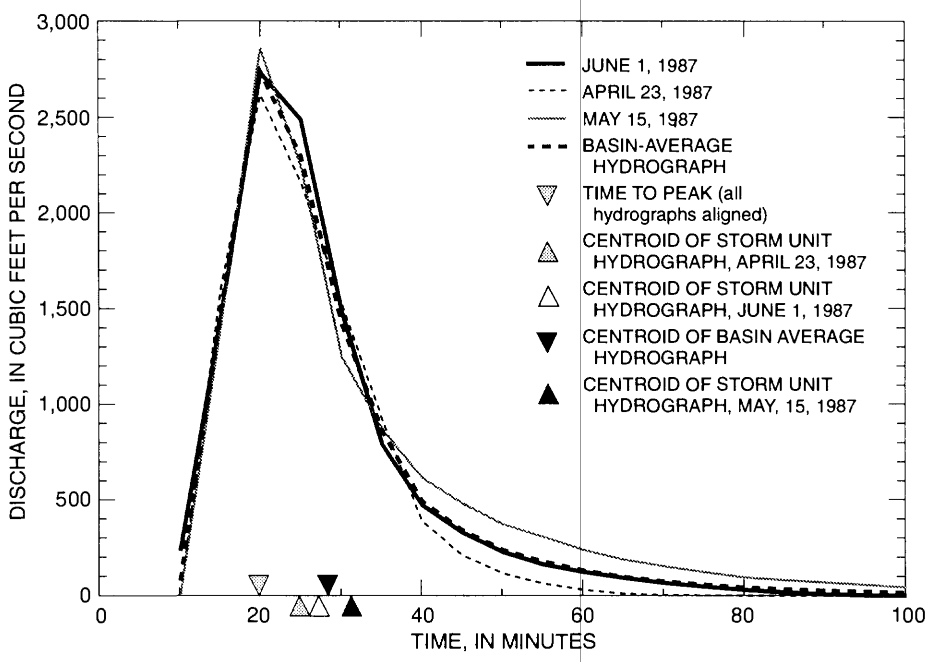

Figure 3. Example of unit hydrographs for selected storms and resulting basin-average unit hydrograph for Nasty Branch at Asheville, North Carolina (site 3).

Table 2. Computation of basin-average unit hydrograph for Nasty Branch (site 3) from storm unit hydrograph

[ft^3/s, cubic feet per second; Storm unit hydrograph peaks are aligned and hydrographs are averaged. Time ordinates of the average hydrograph are corrected by subtracting a time correction equivalent to the difference between the centroid, t_{RO}, of the average hydrograph and the average of the centroids of the storm unit hydrographs. In the case shown, the correction is negated by rounding of the revised time ordinate to the nearest computation interval--5 minutes.

Time ordinate (minutes)	Storm unit hydrograph for June 1, 1987 (ft^3/s)	Storm unit hydrograph for April 23, 1987 (ft^3/s)	Storm unit hydrograph for May 15, 1987 (ft^3/s)	Basin average unit hydrograph (ft^3/s)	Revised time ordinate (minutes)
5	0.00	0.00	0.00	0.00	5
10	232.35	.00	.00	77.45	10
15	1,450.11	1,605.98	1,405.75	1,487.28	15
20	2,731.83	2,617.45	2,860.02	2,736.43	20
25	2,487.21	2,157.06	2,241.96	2,295.41	25
30	1,484.15	1,524.67	1,246.93	1,418.58	30
35	791.87	923.44	870.80	862.04	35
40	470.50	385.42	614.55	490.16	40
45	326.34	207.87	479.72	337.98	45
50	225.50	113.80	370.02	236.44	50
55	160.22	61.62	304.24	175.36	55
60	121.05	29.13	236.67	128.95	60
65	91.14	10.44	185.87	95.82	65
70	65.29	2.92	151.24	73.15	70
75	45.97	1.67	121.11	56.25	75
80	28.74	.00	93.13	40.62	80
85	15.99	.00	79.39	31.79	85
90	5.74	.00	66.86	24.20	90
95	.74	.00	54.87	18.54	95
100	.00	.00	41.23	13.74	100
105	.00	.00	28.77	9.59	105
110	.00	.00	18.54	6.18	110
115	.00	.00	13.89	4.63	115
120	.00	.00	12.93	4.31	120
125	.00	.00	7.54	2.51	125
130	.00	.00	2.22	.74	130
135	.00	.00	1.47	.49	135
140	.00	.00	.40	.13	140
Centroid (minutes)	27.22	25.41	32.04	28.41	

Average centroid of storm unit hydrograph (minutes)	28.22
Time correction (minutes)	.19

Dimensionless Hydrographs

In addition to formulation of unit hydrographs from observed data, methods also have been developed for estimating unit hydrographs for ungaged basins in the absence of data. Two distinct approaches are used to develop these hydrographs. One approach involves the use of mathematical expressions for modeling and estimating unit hydrographs. The other method is based on development and use of dimensionless hydrographs (Commons, 1942; Clark, 1945; U.S. Department of Agriculture, 1972; Stricker and Sauer, 1982). The latter approach is used in this study. A dimensionless hydrograph can be used, with estimates of peak flow and basin lagtime, to synthesize flood hydrographs of ungaged basins.

From an extensive study of the hydrographs of 61 rural basins and 19 urban basins, Inman (1987) derived a dimensionless hydrograph for basins in Georgia (hereafter referred to as the Georgia hydrograph). Comparison of synthetic flood hydrographs produced by use of the Georgia hydrograph with observed flood hydrographs from Georgia streams indicated that the synthetic hydrographs had standard errors of estimate of 39.5 percent at hydrograph widths corresponding to 50 percent of peak flow (w_{50}, fig. 2), and 43.6 percent at widths corresponding to 75 percent of peak flow (w_{75}, fig. 2).

Subsequent investigations have shown that the Georgia hydrograph is applicable to many topographic and hydrologic conditions in the southeastern United States (Robbins, 1986; Olin and Atkins, 1988; Gamble, 1989; Neely, 1989; Becker, 1990; Bohman, 1990). However, observed hydrographs from streams draining low topographic relief basins in western Tennessee and in the Coastal Plain of South Carolina were considerably wider than those estimated by using the Georgia hydrograph (Gamble, 1989; Bohman, 1990). The time bases of hydrographs of streams draining the Blue Ridge were somewhat narrower than those obtained by using the Georgia model (Bohman, 1990). These regional differences led Bohman to develop new dimensionless hydrographs for rural basins in each of the three physiographic provinces of South Carolina.

Several investigations also have determined that the Georgia hydrograph is suitable for application to urban basins in many areas of the southeastern United States (Robbins, 1986; Olin and Atkins, 1988; Gamble, 1989; Neely, 1989; Becker, 1990). However, Bohman (1992) determined that the Georgia hydrograph overestimated the widths of observed hydrographs of urban basins in both the Piedmont and upper Coastal Plain of South Carolina. As a result, Bohman developed new dimensionless hydrographs for urban areas in South Carolina.

Based on the experiences of Bohman in South Carolina, it was hypothesized that a unique dimensionless hydrograph would be required to successfully synthesize flood hydrographs for urban areas of North Carolina. Following the lead of Inman (1987), a regionally averaged unit hydrograph was developed using data collected from urban streams in North Carolina. The new dimensionless hydrograph (hereafter referred to as the North Carolina hydrograph) was developed by aligning the 29 dimensionless, duration-transformed, basin-unit hydrographs by time of peak flow, and averaging flow ordinates for each time step. Nondimensionalization was accomplished by dividing the flow ordinates by the peak flow, and time values by basin lagtime. However, as shown in a subsequent section of this report, testing of the North Carolina hydrograph revealed that it did not apply to streams which drained urban basins in or around Asheville. In an effort to find other methods for synthesizing flood hydrographs, the Georgia and selected South Carolina hydrographs were tested for their applicability to this area and to other regions of the State.

Basin Lagtime and Other Characteristics

Lagtime is often defined as the time elapsed between the occurrence of the center of mass of excess precipitation and the occurrence of the center of mass of the resulting runoff (fig. 2). For this study, the lagtime was computed as the time corresponding to the centroid of the unit hydrograph minus one-half of the computation interval (storm duration) used to produce the unit hydrograph. The two definitions are mathematically equivalent and give the same result. This definition is appropriate for this study because conceptually, the O'Donnell method treats each incremental unit of uniform rainfall as an individual storm of that incremental duration. The basin lagtime was estimated as the average of the lagtimes for the basin.

To provide additional lagtime data, lagtimes developed by Putnam (1972) for basins of less than 15 mi^2 and covered by at least 1 percent impervious area were added to the database (table 1). The addition of the 25 sites from Putnam to the 25 sites for which lagtime was computed during this study resulted in estimates of lagtime for 50 basins in North Carolina (table 3).

Information on selected basin characteristics for the study basins was obtained from a variety of sources (Putnam, 1972; Gunter and others, 1987; Robbins and Pope, 1996). Tabulated basin characteristics, which are defined in the glossary and shown in table 3, include drainage area, stream length, stream slope, percentage of impervious area, basin-development factor, lagtime, and the 2-year 2-hour rainfall amount. Drainage area, as used in this report, refers to the contributing drainage area. In urban areas, storm drainage systems are sometimes constructed to cross topographic divides and thereby divert runoff into or out of the topographically defined basin, which should be accounted for when computing drainage area. The tabulated basin characteristics represent physical, developmental, and climatological conditions in the basin. Linear regression was used to develop a relation between basin lagtime and easily measurable physical characteristics of the basin.

ESTIMATING FLOOD HYDROGRAPHS FOR URBAN BASINS

This section describes the development and testing of the North Carolina hydrograph. Hydrographs synthesized by using the North Carolina hydrograph are compared with measured hydrographs. The applicability of the North Carolina hydrograph is compared with the applicability of previously developed dimensionless hydrographs for Georgia and South Carolina. A relation for estimating basin lagtime from easily determined basin characteristics is presented. Finally, an example showing application of the technique for estimating flood hydrographs for ungaged urban basins in North Carolina is given.

Regionalized Unit Hydrograph for North Carolina Urban Basins

As previously described, the O'Donnell (1960) method for unit hydrograph determination derives unit hydrographs of durations equal to one data recording interval, which was either 5 or 15 minutes for this study. However, in order to regionalize the unit hydrograph, duration is most effectively expressed as some function of basin characteristics, usually basin lagtime. Therefore, following the approach used by Inman (1987), the 29 short-duration, basin unit hydrographs were transformed, by using equation 2, into unit hydrographs with durations equal to selected fractions of the basin lagtime ($0.25L_t$, $0.33L_t$, $0.50L_t$, and $0.75L_t$):

$$q_D(t) = (1/n)[q(t) + q(t\text{-}1) + \ldots + q(t\text{-}n\text{+}1)] \qquad (2)$$

where:

- $q_D(t)$ is the flow ordinate of the unit hydrograph of duration D at time t;
- $q(t)$ is the flow ordinate of the original unit hydrograph at time t; and
- n is the ratio of the desired duration, D, of the unit hydrograph to the data-recording interval. D must be an integer multiple of the data-recording interval.

The transformed basin-averaged unit hydrographs were converted into dimensionless terms by dividing the flow ordinates by the peak flows and the time ordinates by the basin lagtimes. Four sets of 29 dimensionless, transformed basin unit hydrographs resulted from this process.

Four alternative, dimensionless hydrographs--one for each lagtime fraction--were derived from the dimensionless, transformed basin unit hydrographs by averaging the 29 dimensionless, transformed basin unit hydrographs corresponding to that lagtime fraction. Time was measured relative to the time of peak flow. Each of the four dimensionless hydrographs was then tested for fit and bias against observed data to select the one best dimensionless hydrograph for urban basins in North Carolina. The test consisted of using each of the dimensionless hydrographs to develop synthetic flood hydrographs for the 101 observed floods. Synthetic hydrographs were developed by multiplying the flow ordinates of dimensionless hydrographs by the peak flow observed during the flood, and the time ordinates by the basin average lagtime. Subsequently, the widths (w_{50}, w_{75}) (fig. 2) of the resulting synthetic hydrographs were compared to the widths of the observed flood hydrographs. Using this test, the North Carolina hydrograph derived from the original basin unit hydrographs by using equation 2 and a value of $D = 0.33\,L_t$ (table 4) provided lower standard error (39.8 percent) than those derived using the other three values of D. Statewide, the average error between the observed and synthesized hydrographs was 7.3 percent.

Table 3. Selected basin characteristics at sites used in flood hydrographs and lagtime analyses in North Carolina

[mi^2, square mile; mi, mile; ft/mi, foot per mile; BDF, basin-development factor; hr, hour; in., inch; ---, not determined. See glossary for definition of characteristics.]

	Physical characteristics of study basins							
Site number (fig. 1)	Drainage area (mi^2)	Stream length (mi)	Stream slope (ft/mi)	Impervious area (percent)	BDF	Lag-time (hr)	2-yr, 2-hr rainfall (in.)	Number of hydrographs
Asheville sites (Blue Ridge)								
1	2.13	2.27	147	17.5	5	0.92	1.9	2
2	.57	1.36	475	4.5	1	.83	1.9	3
3	1.36	1.98	90	31.4	11	.42	1.9	3
4	2.46	2.91	156	15.0	4	.51	1.9	4
Morganton site (Piedmont)								
6	8.26	6.56	28	3.0	6	3.93	1.9	3
Charlotte sites (Piedmont)								
7	.62	1.33	75	18.0	---	.65	1.9	---
8	.27	1.12	102	19.0	---	.70	1.9	---
9	12.2	8.05	20	25.0	---	1.30	2.0	---
10	9.4	5.55	28	8.0	---	2.00	2.0	---
11	.56	1.10	59	16.0	---	.54	2.0	---
12	.52	1.07	70	20.0	9	.62	1.9	3
13	14.50	7.03	18	8.0	---	2.50	1.9	---
14	2.72	2.66	41	15.0	9	1.28	1.9	2
15	.44	1.05	83	14.0	---	.50	1.9	---
16	6.98	5.06	25	6.0	---	2.99	2.0	---
17	13.00	9.82	14	4.0	---	6.00	2.0	---
Greensboro site (Piedmont)								
18	14.70	6.36	21	2.0	---	3.82	2.0	---
Winston-Salem sites (Piedmont)								
19	10.20	6.70	26	3.0	---	3.50	1.9	---
20	5.25	4.50	30	6.0	---	3.34	1.9	---
21	.89	1.62	88	12.0	5	.62	1.9	2
22	11.90	4.07	46	9.0	---	1.50	1.9	---
23	.55	1.10	143	37.0	9	.31	1.9	3
24	.59	1.27	156	28.0	7	.33	1.9	2
25	5.30	4.39	46	20.0	---	.76	1.9	---
26	9.73	7.16	21	2.0	---	5.24	1.9	---
27	11.80	10.60	28	6.0	---	4.00	1.9	---
28	6.81	6.70	31	7.0	---	2.50	1.9	---

Table 3. Selected basin characteristics at sites used in flood hydrographs and lagtime analyses in North Carolina--Continued

[mi^2, square mile; mi, mile; ft/mi, foot per mile; BDF, basin-development factor; hr, hour; in., inch; ---, not determined. See glossary for definition of characteristics.]

Site number (fig. 1)	Physical characteristics of study basins: Drainage area (mi^2)	Stream length (mi)	Stream slope (ft/mi)	Impervious area (percent)	BDF	Lag-time (hr)	2-yr, 2-hr rainfall (in.)	Number of hydrographs
Durham sites (Piedmont)								
29	.81	1.70	48	32.0	---	.59	2.1	---
31	.52	1.48	72	20.0	---	.47	2.1	---
Raleigh sites (Piedmont)								
32	2.75	2.56	37	4.7	3	2.06	2.1	4
34	.98	1.06	64	10.4	3	1.17	2.1	7
35	.19	.60	127	34.2	6	.29	2.1	6
36	.56	1.48	81	9.0	---	.60	2.1	---
37	.78	1.72	75	14.0	---	.66	2.1	---
38	.27	.61	162	54.6	10	.24	2.1	6
39	.08	.55	121	41.7	9	.26	2.1	3
40	1.09	1.98	89	3.85	3	2.86	2.1	3
42	1.27	1.93	80	32.4	8	1.59	2.1	4
44	.66	1.42	60	10.3	7	.47	2.1	4
Fayetteville sites (Sand Hills)								
45	7.63	6.21	37	1.0	0	13.30	2.1	4
46	.64	.89	27	19.4	4	1.35	2.2	3
47	.82	1.00	20	48.0	11	1.05	2.2	4
48	2.74	3.60	14	26.2	6	5.83	2.2	3
49	.64	1.81	30	27.0	8	.87	2.2	2
50	.10	.28	174	23.0	10	.27	2.2	4
51	.04	.11	375	12.0	4	.49	2.2	3
Goldsboro site (Coastal Plain)								
52	2.17	3.26	11	30.0	4	1.45	2.3	4
Wilson site (Coastal Plain)								
53	7.92	4.92	11	11.0	5	2.33	2.1	4
Greenville site (Coastal Plain)								
54	9.10	4.85	9	2.0	2	8.39	2.3	3
Wilmington site (Coastal Plain)								
55	1.98	1.82	15	6.0	3	2.15	2.6	3

Table 4. Dimensionless time and discharge ratios of the North Carolina, Georgia, and selected South Carolina hydrographs

[t/L_t, q/Q_p, t, time; L_t, basin lagtime; q, flow ordinate; Q_p, peak flow; ---, not available]

Dimensionless time ratios (t/L_t)	North Carolina hydrograph, dimensionless discharge ratios (q/Q_p)	Georgia hydrograph, dimensionless discharge ratios (q/Q_p)	South Carolina upper Coastal Plain urban hydrograph, dimensionless discharge ratios (q/Q_p)	South Carolina Blue Ridge rural hydrograph, dimensionless discharge ratios (q/Q_p)
0.05	---	---	0.07	---
.10	0.06	---	.10	---
.15	.09	---	.15	0.08
.20	.13	---	.21	.14
.25	.18	0.12	.28	.22
.30	.23	.16	.37	.31
.35	.30	.21	.47	.43
.40	.37	.26	.58	.56
.45	.45	.33	.69	.69
.50	.54	.40	.79	.80
.55	.64	.49	.87	.89
.60	.73	.58	.93	.96
.65	.82	.67	.97	.99
.70	.89	.76	1.00	1.00
.75	.94	.84	.97	.97
.80	.97	.90	.94	.93
.85	1.00	.95	.89	.88
.90	.97	.98	.83	.82
.95	.94	1.00	.77	.76
1.00	.89	.99	.71	.71
1.05	.85	.96	.65	.65
1.10	.80	.92	.59	.60
1.15	.75	.86	.54	.56
1.20	.70	.80	.49	.51
1.25	.65	.74	.44	.47
1.30	.60	.68	.40	.44
1.35	.55	.62	.37	.41
1.40	.51	.56	.34	.38
1.45	.47	.51	.31	.35
1.50	.43	.47	.28	.33
1.55	.40	.43	.26	.30
1.60	.37	.39	.24	.28
1.65	.34	.36	.22	.26
1.70	.31	.33	.20	.24
1.75	.29	.30	.19	.23
1.80	.26	.28	.17	.21
1.85	.24	.26	.16	.20
1.90	.22	.24	.15	.19
1.95	.21	.22	.14	.17
2.00	.19	.20	.13	.16
2.05	.18	.19	.12	.15
2.10	.16	.17	.11	.14
2.15	.15	.16	.11	.14
2.20	.14	.15	.10	.13
2.25	.13	.14	.09	.12
2.30	.12	.13	.09	.12
2.35	.11	.12	.08	.11
2.40	.10	.11	.07	.10

However, there were significant regional differences in the distribution of the width errors. For example, the widths (w_{50}) of the synthetic hydrographs for the Asheville sites exceeded the corresponding observed hydrograph widths by an average of 41.2 percent (table 5). This apparent bias was very consistent; the widths of 11 of the 12 observed hydrographs collected in the Asheville area were overestimated. Six of the 12 widths were overestimated by more than 40 percent. On the basis of these observations, application of the North Carolina hydrograph to urban basins in Asheville would likely result in synthetic hydrographs that are wider (or have a longer duration) than the actual widths (actual duration) of hydrographs of streams in that city. Of course, the errors in the widths of the synthetic hydrographs are not solely attributable to the use of the dimensionless hydrographs. Although the observed peak flows were used in the syntheses of the synthetic hydrographs, only the average of the flood hydrograph lagtimes were used, and the use of a basin average rather than the observed values could contribute a significant source of error. Additionally, the basin average lagtime for the Asheville sites was only about 0.7 hour, far less than the average of any other area. Thus, errors caused by rounding the results to the nearest data-recording interval during the testing of the dimensionless hydrographs could have contributed proportionately more to the reported errors in Asheville than in other study areas.

Although the sample of study basins is small (four sites), the consistency and magnitude of the overprediction lends credibility to this finding of bias. However, the extent of this apparent bias in other areas of the Blue Ridge Province cannot be determined without additional data. This regional bias could be addressed by developing a unique dimensionless hydrograph for the Asheville or possibly Blue Ridge area, but additional data would be required before such an approach would be feasible.

For the Coastal Plain study basins (table 5), the widths of the synthetic hydrographs underestimated the widths of the corresponding observed hydrographs by an average of -10.2 percent. However, this error was not statistically different from zero at the 95-percent confidence level.

Comparison of North Carolina, South Carolina, and Georgia Hydrographs

In order to evaluate other options for synthesizing hydrographs, especially for the Asheville area, the Georgia hydrograph (Inman, 1987), the South Carolina Blue Ridge rural hydrograph (Bohman, 1990), and the South Carolina Piedmont and upper Coastal Plain urban hydrographs (Bohman, 1992) were used to estimate flood hydrographs for North Carolina urban basins (table 4). Comparisons of synthetic and measured hydrographs were done in the same manner as was done for the North Carolina hydrograph.

Overall, the hydrographs for North Carolina urban basins synthesized by using the Georgia hydrograph (table 5) compared favorably with the observed data. For widths at 50 percent of peak flow, the standard error of estimate was 40.2 percent, which is slightly higher than the results from the North Carolina hydrograph. The average error across all basins in North Carolina was only 2.1 percent, which indicates no general geographic bias.

However, as with the North Carolina hydrograph, the Georgia method overestimated hydrograph widths for the Asheville basins. The average difference between the measured and predicted widths was 34.9 percent (compared to 41.2 percent for the North Carolina hydrograph), with overestimation of 11 of the 12 measured hydrographs. Half of the w_{50} widths were overestimated by more than 40 percent. The average difference between the measured and predicted w_{75} width was 40.2 percent.

Bohman (1990) also reported that application of the Georgia hydrograph to rural areas of the Blue Ridge in South Carolina resulted in overestimation of observed data. Bohman (1990) developed a new dimensionless hydrograph for application to rural basins of the Blue Ridge, and later (Bohman, 1992) defined dimensionless hydrographs for urban basins in the Piedmont-upper Coastal Plain and lower Coastal Plain of South Carolina. (No data were available for urban basins in the Blue Ridge area of South Carolina.)

The South Carolina Blue Ridge rural hydrograph and the Piedmont-upper Coastal Plain urban hydrograph were applied to the Asheville study basins. The widths of the resulting synthetic hydrographs were compared to the observed data (table 5). The two South

Table 5. Results of application of four dimensionless hydrographs

[NC, North Carolina hydrograph; GA, Georgia hydrograph; SC-R, South Carolina hydrograph for rural Blue Ridge basins; SC-U, South Carolina hydrograph for urban Piedmont and upper Coastal Plain basins]

Region	Number of basins	Number of hydro-graphs	Average error (percent)				Standard error (percent)				Number of basins with more than half of hydrograph widths overestimated				Number of basins with more than half of hydrograph widths underestimated			
			NC	GA	SC-R	SC-U	NC	GA	SC-R	SC-U	NC	GA	SC-R	SC-U	NC	GA	SC-R	SC-U
w_{50}, width of hydrograph at 50 percent of peak flow (fig. 2)																		
Statewide	29	96	7.3	2.1	-4.2	-7.4	39.8	40.2	42.7	90.5	14	13	10	12	11	13	17	5
Blue Ridge (Asheville)	4	12	41.2	34.9	24.5	20.8	41.9	37.8	31.3	29.2	4	4	4	4	0	0	0	0
Piedmont	14	52	6.9	2.8	-3.5	-23.2	38.6	38.9	42.0	42.1	6	5	3	3	6	8	11	0
Sand Hills (Fayetteville)	7	18	-.7	-8.7	-12.6	-10.5	52.4	55.0	57.9	25.8	3	3	2	2	2	3	3	4
Coastal Plain	4	14	-10.2	-14.2	-20.8	-18.2	27.2	30.0	36.2	38.8	1	1	1	3	2	2	3	1
w_{75}, width of hydrograph at 75 percent of peak flow (fig. 2)																		
Statewide	29	96	6.7	6.6	-3.1	-6.6	43.8	43.9	47.4	49.2	14	13	9	13	13	13	15	13
Blue Ridge (Asheville)	4	12	40.3	40.2	23.6	21.9	42.9	42.8	33.1	32.2	4	4	3	3	0	0	0	0
Piedmont	14	52	5.7	6.2	-4.0	-8.1	43.6	43.6	48.0	47.1	7	6	6	5	7	7	8	8
Sand Hills (Fayetteville)	7	18	.1	-1.4	-4.7	-4.0	56.8	58.04	62.4	59.4	2	2	2	2	4	4	4	4
Coastal Plain	4	14	-9.9	-10.0	-20.7	-21.8	30.2	30.2	38.8	39.4	1	1	1	3	2	2	3	1

Carolina hydrographs overestimated Asheville basin hydrographs, but errors were smaller than those obtained using the North Carolina and Georgia hydrographs (table 5). Values of w_{50} estimated by using the South Carolina rural Blue Ridge hydrograph were overestimated by an average of 24.5 percent, and those obtained by using the South Carolina Piedmont-upper Coastal Plain urban hydrograph were overestimated by an average of 20.8 percent. These errors were statistically different from zero at the 95-percent confidence level.

The North Carolina and Georgia hydrographs satisfactorily reproduced flood hydrographs measured in the Piedmont, Sand Hills, and Coastal Plain without significant bias. However, both of the hydrographs substantially and consistently overestimated flood hydrographs for the Asheville basins included in this study. The two South Carolina hydrographs tested (rural Blue Ridge and urban Piedmont-upper Coastal Plain) also overestimated hydrographs for Asheville basins, but errors were smaller than those obtained by using the North Carolina and Georgia methods. Based on these results, the North Carolina hydrograph applies to all areas of the State except the Blue Ridge, where the South Carolina urban hydrograph is more applicable.

Relation for Estimating Lagtime at Ungaged Basins

The relation of basin lagtime to selected physical characteristics of the study basins was evaluated using linear regression to develop an equation for estimating the lagtime of ungaged basins. Explanatory variables included stream length, stream slope, percentage of the basin covered by impervious surfaces, basin-development factor, and the 2-year 2-hour storm rainfall amount (table 3). Regressions were performed on log-transformed variables to improve the linearity of the regression fit and to ensure equal distribution of variance for each variable about the regression line. The best-fit relation was in the following form:

$$L_t = 23.2 L^{0.20} S^{-0.52} IA^{-0.50} \qquad (3)$$

where:

L_t is the basin lagtime, in hours;

L is the stream length, in miles;

S is the stream slope, in feet per mile; and,

IA is the percentage of the basin covered by impervious surfaces.

The range of values used to develop equation 3 are as follows: basin lagtime, 0.24 (site 38) to 8.39 (site 54) hours; stream length, 0.28 (site 50) to 10.6 (site 27) miles; stream slope, 9 (site 54) to 162 (site 38) feet per mile (ft/mi); and percentage of basin impervious area, 2.0 (sites 18, 26, and 54) to 54.6 (site 38) percent (table 3).

Five sites were omitted from the final regression analysis (sites 2, 42, 45, 48, and 51). These sites were outliers in one or more basin characteristics and exerted undue influence on the model fit as indicated by Cook's D (Belsley, and others, 1980; Helsel and Hirsch, 1992) and other measures of statistical influence.

The coefficient of determination for equation 3, which indicates the proportion of the total variation of the response variable (lagtime, in this case) that is explained by the regression relation, is 0.90. The standard error for equation 3, which is the range (plus and minus) about the line of the regression that encompasses about two-thirds of the data points, is 31 percent.

The lagtime relation was tested for variable and geographic bias. The presence of variable bias indicates that additional explanatory variables are required in the regression relation or that the form of the relation is incorrectly specified. For this study, variable bias was tested by plotting the residuals of the relation against each of the explanatory variables and examining the plots for consistent relations, patterns, or groupings; none were found. The relation does not appear to be biased with respect to any of the explanatory variables.

Geographic bias indicates that the lagtime relation systematically over- or underestimates the lagtime in one or more subregions of the State. Geographic bias was tested by plotting the residuals of the lagtime relation on a map of North Carolina. The map was reviewed for possible clusters, patterns, or trends; none were detected.

A sensitivity analysis of equation 3 also was performed by evaluating the relative effects of errors in estimates of the explanatory variables on the predicted lagtimes. The analysis was performed by varying estimates of each of the explanatory variables by increments of 5 percent from its respective mean, while holding the other variables constant, and computing the resulting departure from the mean lagtime. The sensitivity analysis indicates that the lagtime relation is most sensitive to errors in computed stream slope and impervious area and least sensitive to errors in estimates of stream length (fig. 4).

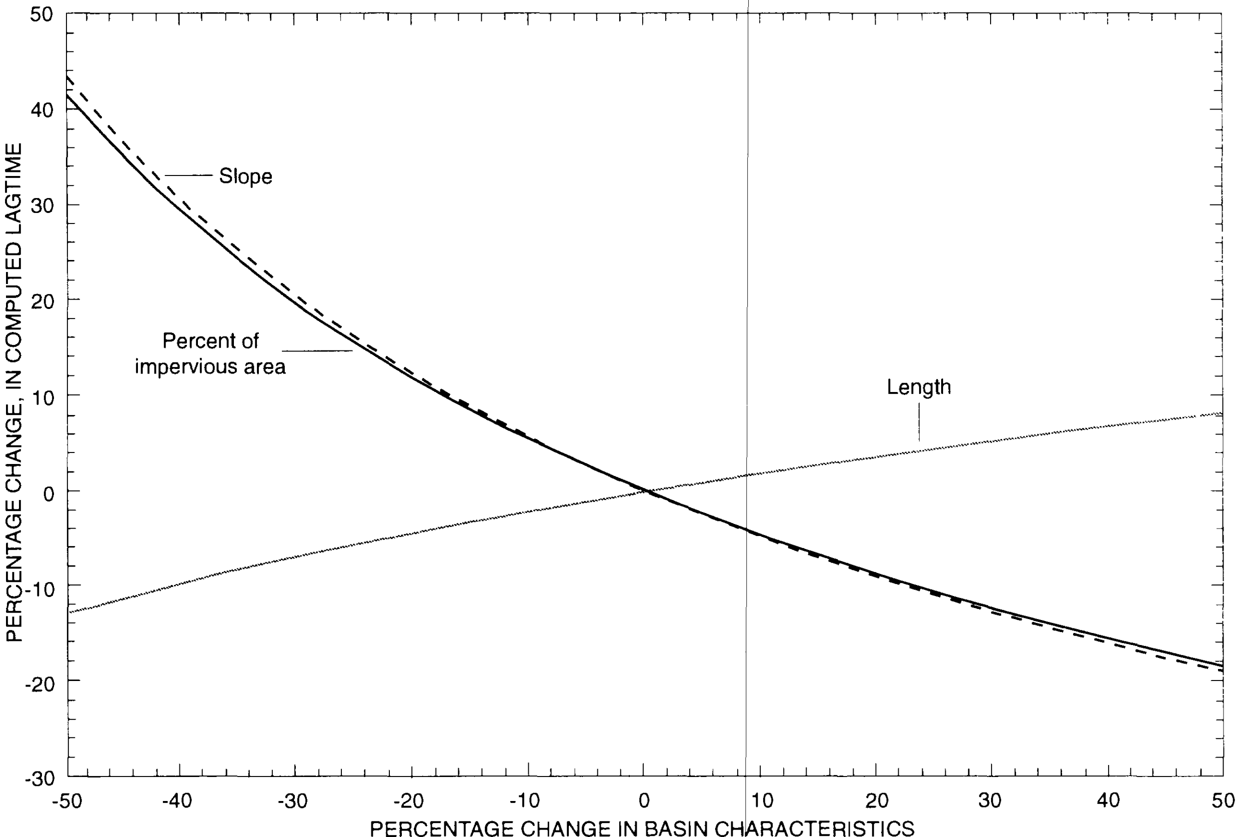

Figure 4. Sensitivity of lagtime relation to errors in estimates of basin characteristics.

Example of Flood Hydrograph Estimation

Application of the flood hydrograph estimation technique requires an estimate of the design peak flow, the basin lagtime, and dimensionless hydrograph. Estimates of peak flows for selected recurrence intervals in ungaged urban basins can be determined by using equations given by Robbins and Pope (1996). Basin lagtime can be determined by using equation 3. Discharge ordinates for the design hydrograph are computed by multiplying the dimensionless flow ratio of the North Carolina or South Carolina hydrograph (table 4) by the design peak flow. The time values of the design hydrograph are determined by multiplying the dimensionless time ratios of the unit hydrograph by the basin lagtime.

As an example application of the procedure, the hydrograph for a flood having a recurrence interval of 25 years will be determined for Richlands Creek near Westover, North Carolina (site 34, table 3). From Robbins and Pope (1996), the peak flow for the 25-year event is estimated as

$$Q_{25} = 28.5\mathrm{D}A^{0.390} IA^{0.436} RQ_{25}{}^{0.338} \quad (4)$$

where:

Q_{25} is the urban basin peak flow having a 25-year recurrence interval;

DA is the basin drainage area, in square miles;

IA is the impervious area, in percent; and

RQ_{25} is the rural basin peak flow having a 25-year recurrence interval, and is given for the Blue Ridge and Piedmont Provinces in Gunter and others (1987) as

$$RQ_{25} = 467DA^{0.655} \quad (5)$$

The drainage area of Richlands Creek is 0.98 mi^2, and the impervious area percentage is 10.4. Consequently, the design flow (in this case, 25-year recurrence interval) is 624 cubic feet per second (ft^3/s). The basin lagtime is computed by using equation 3, where L = 1.06 mi, S=64 ft/mi, and IA = 10.4 percent (table 3), giving L_t = 0.84 hours. (In an application to an ungaged basin, the drainage area, main channel length and slope, and the impervious area percentage would be determined from published or mapped information.)

Finally, the synthesized flood hydrograph (fig. 5) is computed by using the estimates of peak flow (624 ft^3/s), basin lagtime (0.84 hour), and the information in table 4. For example, the first flow ordinate is 37.4 ft^3/s (0.06 x 624 ft^3/s), which occurs at a time of 0.08 hour (0.10 x 0.84 hour) after the beginning of runoff.

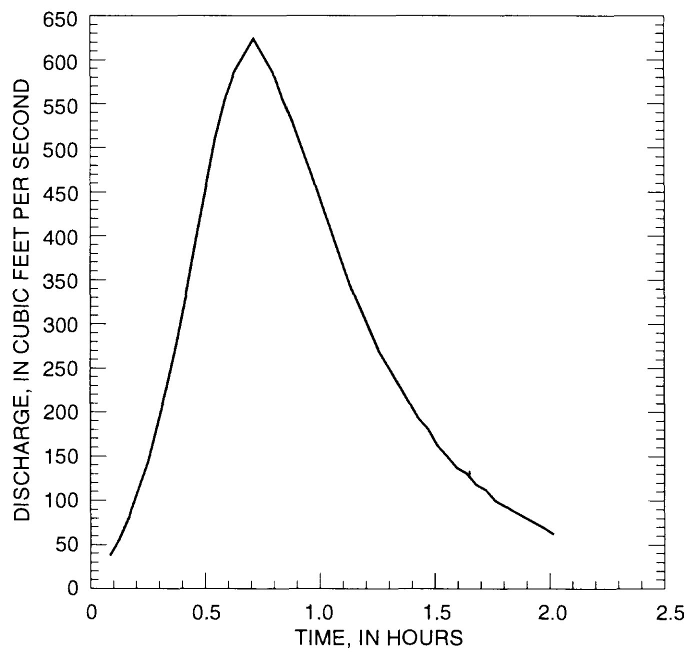

Figure 5. Synthesized flood hydrograph for a 25-year flood on Richlands Creek near Westover, North Carolina (site 34).

SUMMARY

From 1988 to 1993, an investigation was conducted to measure runoff characteristics of small urban basins in North Carolina and to develop techniques for estimating peak flows and storm hydrographs for ungaged urban basins in the State. The study included measurement of streamflow and precipitation at 24 sites and compilation of existing data at 31 additional sites. Data from many of these and other selected urban basins in North Carolina were used to develop methods for estimating peak flows of selected recurrence intervals at ungaged North Carolina urban basins and to develop techniques for estimating flood hydrographs at urban basins in North Carolina.

The flood hydrograph estimation technique described in this report was based on the dimensionless hydrograph method. The development of the dimensionless hydrograph involved five steps: (1) estimating storm unit hydrographs from rainfall-runoff data from 23 urban and 6 urbanizing basins, including streams in Asheville (Blue Ridge Province), Charlotte and Raleigh (Piedmont Province), Fayetteville (Sand Hills region), and selected cities in the Coastal Plain; (2) averaging the storm unit hydrographs to estimate a basin average unit hydrograph for each basin; (3) transforming each basin unit hydrograph into four dimensionless basin unit hydrographs having durations equivalent to each of four lagtime fractions; (4) regionally averaging the duration-transformed basin unit hydrographs to develop four alternate dimensionless hydrographs; and, (5) testing synthetic hydrographs that were developed by application of each of the four alternative dimensionless hydrographs against observed data in order to identify the one best dimensionless hydrograph--the North Carolina hydrograph.

Tests revealed that the North Carolina hydrograph satisfactorily reproduced flood hydrographs measured in the Piedmont, Sand Hills, and Coastal Plain without significant bias. The overall standard error was 39.8 percent, and the average error was 7.3 percent. However, application of the North Carolina hydrograph in the Asheville basins substantially and consistently overestimated observed storm hydrographs. In an effort to identify alternative dimensionless hydrographs for application to the Asheville area, a dimensionless hydrograph developed for Georgia and two dimensionless hydrographs developed for South Carolina were tested. Application of these three dimensionless hydrographs also resulted in synthetic hydrographs that overestimated observed data. Of the three, the South Carolina urban Piedmont-

upper Coastal Plain dimensionless hydrograph had the smallest standard error (29.2 percent) and average error (20.8 percent).

A relation for estimating basin lagtime at ungaged basins from selected basin characteristics also was developed from data collected in 50 urban basins in the State. Lagtime was found to be a function of stream length, stream slope, and the percentage of basin impervious area. Predictions of lagtimes were more sensitive to errors in computed stream slope and impervious area than to errors in estimated stream length.

The techniques described in this report are appropriate for estimating single-peaked hydrographs in urban or urbanizing basins in North Carolina. Basins to which the techniques are applied should have no storage, such as detention ponds, to reduce peak flows. Basin characteristics should generally be within the range of those used to develop the relation.

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GLOSSARY

The following are definitions for selected acronyms and terms used in this report; however, they are not necessarily the only definitions for these acronyms and terms.

Basin-development factor (BDF)--An index of the prevalence of the drainage aspects of storm sewers, channel improvements, impervious channel linings, and curb-and-gutter streets. The range of BDF is from 0 to 12. A value of zero indicates the above drainage aspects are not prevalent, but does not necessarily mean the basin is non-urban. A value of 12 indicates full development of the drainage aspects throughout the basin. Sauer and others (1983) describe how to compute the BDF. In this report, BDF was determined only for basins used in hydrograph analysis.

Channel length--The length, in miles, of the main channel from the basin outlet to the basin divide.

Channel slope--The main channel slope, in feet per mile, as measured from points that are 10 percent and 85 percent of the main channel length as measured from the basin outlet.

Coefficient of determination--The proportion of the total variation of the response variable that is explained by the regression relation.

Dimensionless hydrograph--A unit hydrograph derived by dividing the flow ordinates of the unit hydrograph by the peak flow of the unit hydrograph and the time ordinates by the basin lagtime. The expression, as used in this report, also applies to the regional average of the basin unit hydrographs for the study area.

Drainage area (DA)--The area, in square miles, of a basin that contributes drainage to a stream measured in a horizontal plane. It is usually computed by a planimeter, digitizer, or grid method from U.S. Geological Survey 7.5-minute topographic quadrangle maps. In urban areas, drainage systems sometimes cross topographic divides; such diversions should be accounted for when computing the drainage area.

Impervious area (IA)--The percentage of the drainage basin covered by impervious surfaces, such as houses and other buildings, streets, sidewalks, and parking lots.

Lagtime--Time, in hours, from the occurrence of the center-of-mass of rainfall excess to the occurrence of the center-of-mass of the corresponding runoff. For the purposes of this report, lagtime is computed as the time of the center-of-mass minus one-half of the computation interval (storm duration) used to produce the unit hydrograph.

Peak flow--The maximum discharge, in cubic feet per second, associated with an observed or estimated flood hydrograph.

Standard error of regression--A measure of error, in percent, associated with estimating a streamflow characteristic of a site used in the regression analysis. Approximately two-thirds of the data used in the regression analysis lies within one standard error of the fitted regression relation.

Unit hydrograph--A hydrograph of direct runoff resulting from 1 inch of excess rainfall uniformly generated over the basin at a uniform rate during a specified period of time or duration (Chow, 1964).

Urban basin--A basin for which the basin-development factor (BDF) is generally greater than 3.

Urbanizing basin--An urbanizing basin is defined by the presence of impervious surfaces which cover at least 1 percent of the basin land area and drainage improvements sufficient to warrant estimation of the basin development factor of at least 1, but no more than 3.

2-year, 2-hour rainfall--The amount of rainfall, in inches, for the 2-year, 2-hour storm as reported in Hershfield (1961).