SIMULATION OF FLOOD HYDROGRAPHS FOR SMALL BASINS IN MISSOURI

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CONVERSION FACTORS

Multiply	By	To obtain
inch	25.40	millimeter
square foot	0.09294	square meter
cubic foot	0.02832	cubic meter
mile	1.609	kilometer
square mile	2.590	square kilometer
foot per second	0.3048	meter per second
cubic foot per second	0.02832	cubic meter per second
foot per mile	0.1894	meter per kilometer
acre-foot	1,233	cubic meter

GLOSSARY

Basin development factor (BDF)--An index of the prevalence of the drainage aspects of: (1) Channel improvements, (2) impervious channel linings, (3) storm sewers, and (4) curb-and-gutter streets in a drainage basin. This index has a range of 0 to 12. A value of zero indicates the above drainage aspects are not prevalent, but does not necessarily mean that the basin is nonurban. A value of 12 indicates full development of the drainage aspects throughout the basin. See "Supplemental Data" at the back of this report for details pertaining to computing BDF.

Basin lagtime, or lagtime, (LT)--The elapsed time, in hours, from the centroid of rainfall excess to the centroid of the resultant runoff hydrograph (Inman, 1987, p. 10). Lagtime is computed from the unit hydrograph.

Basin length (L)--The basin length, in miles, is measured on topographic maps along the main channel from the streamflow-gaging station or other site of interest to the basin divide.

Cubic feet per second--The rate of discharge; 1 cubic foot per second is the rate of discharge of a stream having a cross-sectional area of 1 square foot and an average velocity of 1 foot per second:

1 cubic foot per second is approximately equal to 1.9835 acre-feet per day.

Drainage area (A)--The contributing drainage area, in square miles, is determined by delineating the drainage-basin boundary on topographic maps and planimetering the area within the boundary. In urban areas, drainage systems may cross topographic divides and such changes need to be accounted for when computing drainage area.

Flood frequency--The relation between return period or recurrence interval, in years, and flood-peak magnitude, in cubic feet per second.

Flood hydrograph--A graphical representation of the fluctuation in flow (in cubic feet per second) in a stream with respect to time.

Flood-peak discharge (Q_p)--The maximum discharge during a flood.

Flood volume (V)--The runoff, in acre-feet, either computed by summing the discharge ordinates at a given time interval for the flood hydrograph and converting the sum to acre-feet or estimated by using a regression equation.

Impervious area (I)--The percentage of the contributing drainage area that is nonpervious because of buildings, streets and roads, parking lots, and other impervious areas within an urban basin. A procedure for determining the percentage of impervious area is described by Spencer and Alexander (1978, p. 5). Impervious area may be estimated using an alternative basin characteristic (Southard, 1986).

Main-channel slope (S)--Main-channel slope, in feet per mile, is the average slope between points 10 and 85 percent of the distance along the main stream channel from the site to the basin divide.

Recurrence interval--As applied to floods, recurrence interval is the average number of years within which a given flood peak will be equaled or exceeded once. For example, the discharge of a 100-year flood will be equaled or exceeded on the average of once in 100 years. In terms of probability, there is a 1-percent chance that such a flood will occur in any year.

Streamflow-gaging station--A gaged site where a record of discharge of a stream is obtained. Also, concurrent records of precipitation might be collected at streamflow-gaging stations operated for special projects or studies.

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ABSTRACT

A dimensionless hydrograph for use in simulating flood hydrographs for small rural and urban basins in Missouri has been developed by the U.S. Geological Survey. Development of the dimensionless hydrograph included computing: (1) Unit hydrographs and basin lagtimes for 341 floods recorded at 41 streamflow-gaging stations located along small rural and urban streams in Missouri, (2) an average unit hydrograph and an average basin lagtime for each station; and (3) unit hydrographs of one-fourth, one-third, one-half, and three-fourths duration of the average basin lagtime from the average unit hydrograph for each station. Dimensionless hydrographs then were obtained by dividing coordinates of discharge by peak discharge and of time by basin lagtime. Recorded data were best described by a dimensionless hydrograph based on a duration of one-half basin lagtime. An average dimensionless hydrograph applicable to both rural and urban basins was developed by averaging the dimensionless hydrographs determined for each of the 41 gaged sites.

Hydrograph widths for various ratios of discharge versus peak discharge are given for the dimensionless hydrograph developed for Missouri. Hydrographs were simulated and differences in simulated and actual hydrograph widths at 50- and 75-percent of the peak discharge were computed and statistically analyzed. Standard errors of estimate of ±37.8 percent for 50-percent of peak-discharge width and ±42.6 percent for 75-percent of peak-discharge width were determined for single-peak hydrographs.

A technique incorporating the dimensionless hydrograph is defined for simulating flood hydrographs for small rural and urban basins in Missouri. Flood hydrographs associated with future flood-peak discharges resulting from rainfall-induced runoff can be simulated, and estimates of basin lagtime and flood-runoff volume can be made. This technique was developed from an analysis of flood records for 61 streamflow-gaging stations in small basins in Missouri.

Final hydrograph shape and flood-runoff-volume analyses are based on a balanced, representative sampling of data from 41 of the 61 gaged sites in Missouri. This sample included 24 rural sites and 17 urban sites statewide. Sixty-one gaged sites (27 rural and 34 urban) were used in analysis of basin lagtime. Multiple-regression analyses were used to relate basin lagtimes and flood-runoff volumes to selected drainage-basin characteristics. Also, equations are provided, as supplemental data, for estimating the peak discharge of floods in rural and urban basins having a 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval.

Alternative methods are provided for estimating flood-runoff volumes at ungaged sites by using either regression or numerical- integration equations that require determination of the basin lagtime and peak discharge. The average standard error of estimate for flood-runoff volume based on regression equation is ±32.3 percent.

Flood-hydrograph simulation, basin-lagtime estimation, and flood-runoff-volume estimation procedures and equations are considered applicable to ungaged sites in basins having drainage areas of about 0.25 to 40 square miles. These procedures and equations are applicable to flood flows that are not significantly affected by storage or diversions.

INTRODUCTION

Because flooding remains a major problem nationwide (Becker, 1985), flood flows from rural and urban basins need to be considered in: (1) Designing street and highway structures, such as bridges and culverts; (2) land-use planning; (3) establishing rates for flood insurance; and (4) formulating emergency evacuation plans for flood-prone areas. There is a continuing need to evaluate the flood-related risks associated with the design of highway culverts and bridges. Such risks include interruption of traffic and encroachment of floodwater into the upstream flood plain, as well as monetary losses because of damages to the roadway and the drainage structure. Flood hydrographs are necessary to determine the water-surface elevation at and upstream from the roadway, and to estimate the duration of inundation. Because many culverts and bridges are located at ungaged sites, simulated flood hydrographs are commonly required.

Urbanization in a drainage basin results in changes in flood-flow characteristics in the drainage basin. These changes usually include increased peak discharges because of increased impervious area, and decreased basin lagtime for basins that do not have substantial in-channel or detention storage (Sauer and others, 1983). A report by Stricker and Sauer (1982) provides techniques for estimating flood hydrographs for ungaged urban basins throughout the United States.

The U.S. Geological Survey, in cooperation with the Missouri Highway and Transportation Commission, conducted an investigation to determine characteristics of flood hydrographs from small gaged rural and urban drainage basins, and to develop a technique for simulating flood hydrographs at ungaged sites in Missouri. This information can be used for risk analysis (Corry and others, 1980) of highway drainage structures. The primary objective of this investigation was to provide highway engineers and other designers with a reliable technique to simulate the flood hydrographs and to estimate flood-runoff volumes that can be expected to occur in small rural and urban basins in Missouri.

Purpose and Scope

This report summarizes the data and analytical procedures used in the investigation. The report also presents regression equations developed for estimating basin lagtimes, peak discharges, and flood volumes, and describes a technique for simulating flood hydrographs. Descriptions of the applicability, accuracy, and limitations of the equations and technique, and examples of their use are given. This is the final report resulting from the investigation of flood hydrographs from small rural and urban basins in Missouri, and supplements a previous report (Becker, 1986) that provides techniques for estimating flood-peak discharges from urban basins.

Approach

Several previously documented methods for simulating flood hydrographs were investigated for possible use in this statewide study. These general methods included the Commons (1942) method, Clark (1945) method, and U.S. Department of Agriculture, Soil Conservation Service (1972) method, among others. A method used by Becker (1980), based on earlier work by Commons (1942), and Craig and Rankl (1978), adequately described flood-hydrograph shapes in South Dakota. It was thought that this modified Commons (1942) method also might be applicable to small basins in Missouri. The Clark (1945) method was used by Stricker and Sauer (1982) to develop a dimensionless hydrograph that can be used to estimate flood hydrographs for ungaged urban basins throughout the United States. Applicability of the Clark (1945) method to 25 small urban streams in St. Louis County, Missouri was demonstrated by Stricker and Sauer (1982).

The approach of this study involved testing and comparing these and other methods for applicability to small basins in Missouri based on fitting model-simulated flood hydrographs to actual flood hydrographs in dimensionless form. A computer model to calculate and plot flood hydrographs in dimensionless form was developed for testing the Commons (1942), Clark (1945), and U.S. Department of Agriculture, Soil Conservation Service (1972) methods. After consideration of these methods, a simulation technique developed by the U.S. Geological Survey for basins in Georgia

(Inman, 1987) was considered. Because Inman's (1987) dimensionless hydrograph was developed and tested for a variety of conditions (including urban, rural, mountainous, coastal plain, and small and large drainage basins), it was theorized that this dimensionless hydrograph also would be applicable to basins in Missouri.

The simulation technique developed for basins in Georgia (Inman, 1987, p. 2-6) proved most useful and provided a more reliable result than did the other methods investigated because of a more rigorous analytical procedure. Computer programming utilized by Inman (S.E. Ryan, U.S. Geological Survey, written commun., 1986) was adapted for use in Missouri. Unit hydrographs, based on the O'Donnell (1960) method, and basin lagtimes are computed from recorded rainfall and discharge data for gaged sites.

DATA BASE

The U.S. Geological Survey, in cooperation with the Missouri Highway Commission (now Missouri Highway and Transportation Commission), began collecting hydrologic data from 43 streamflow-gaging stations (Hauth, 1973) on small rural streams throughout Missouri during 1948. The data-collection emphasis of the small-streams program was changed in 1976 from rural to urban areas of Missouri with the establishment of 11 streamflow-gaging stations to sample rainfall and runoff from urban basins throughout Missouri. Hauth (1980) determined that further data collection on small rural streams in Missouri would not appreciably improve available flood-frequency regression models. In 1970, the U.S. Geological Survey, in cooperation with St. Louis County, began to collect and analyze data necessary to define the effects of urban development on surface runoff from 30 small drainage basins in St. Louis County (Spencer and Alexander, 1978). Data collected at these gaged sites provide the basis for transferability of flood data to ungaged small basins throughout Missouri.

An investigation of peak discharges by Hauth (1974a) provided data necessary for analysis of flood hydrographs from small basins in the rural setting. Investigations of peak discharges from urban sites (Spencer and Alexander, 1978, and Becker, 1986) provided the necessary data for analysis of flood hydrographs from small basins in the urban setting. A representative data base was selected from the large quantity of data available. The locations of the 61 streamflow-gaging stations for which rural and urban data were considered in this study are shown in figures 1 and 2. Basin characteristics for these gaging stations are listed in table 1.

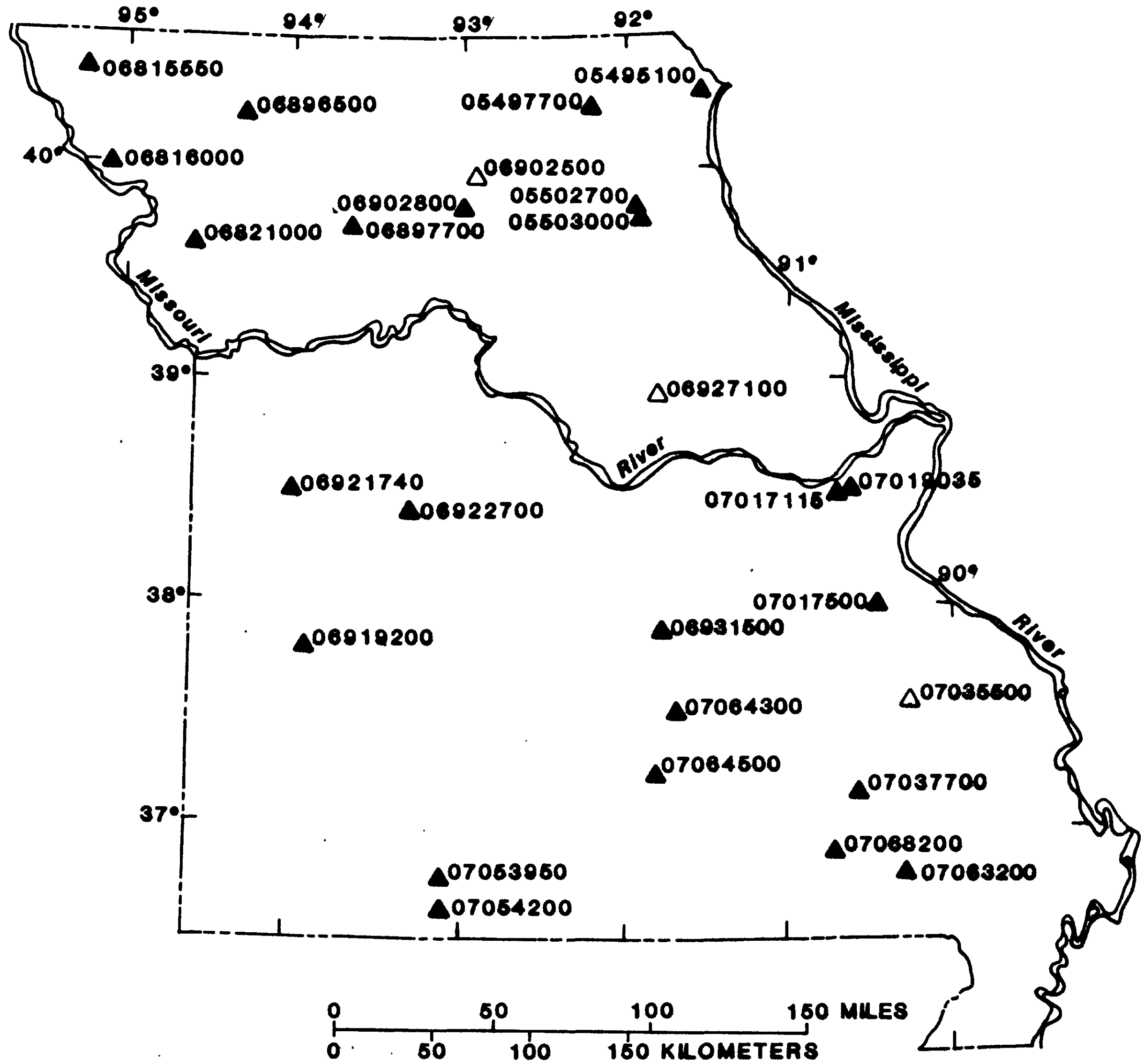
Data used in this study includes that for 27 rural sites statewide (Hauth, 1974a), 25 urban sites in St. Louis County (Spencer and Alexander, 1978), and 9 urban sites statewide (Becker, 1986). Flood hydrographs considered for each of these sites numbered from about 5 to about 30.

ANALYTICAL PROCEDURES

Because of the large data base available, specialized computer programs were developed to determine which was the better hydrograph-simulation method to describe flood hydrographs in Missouri. These programs provided the means of comparing simulated and actual flood hydrographs of varied magnitude for regional groups of sites based on differing methods.

The study included evaluation of hydrograph-simulation methods, development and testing of computer programs, and data analyses. Techniques for both hydrograph-shape and flood-runoff-volume estimation were developed. Alternative methods used by Becker (1980) and Stricker and Sauer (1982) for hydrograph simulation were tested. However, the dimensionless-hydrograph method developed by the U.S. Geological Survey in Georgia (Inman, 1987) was modified as necessary and adopted for use in the this investigation.

To analyze all station data would be extremely time consuming, so the analytical procedure was that of a sampling procedure. Some of the data were used to develop a hydrograph-simulation technique and selected data from the remaining data base were used for verification and error-analysis comparisons. Detailed analyses of these hydrographs for rural and urban gaged sites were made using the Inman (1987) method.

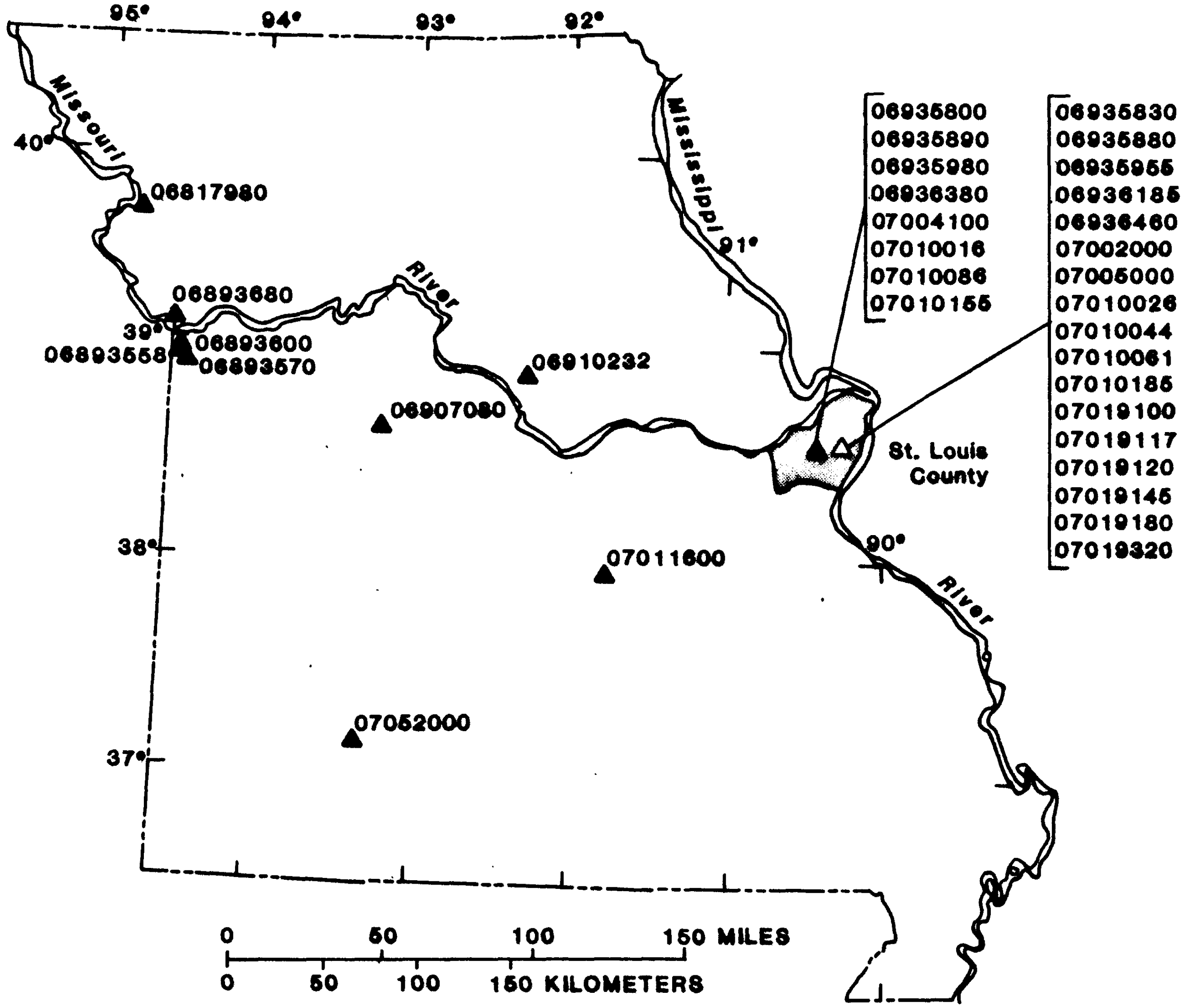


EXPLANATION

06816000▲ U.S. GEOLOGICAL SURVEY STREAMFLOW-GAGING STATION USED IN DIMENSIONLESS HYDROGRAPH DEVELOPMENT AND REGRESSION ANALYSES

06902500△ U.S. GEOLOGICAL SURVEY STREAMFLOW-GAGING STATION USED ONLY IN REGRESSION ANALYSES

Figure 1.--Location of streamflow-gaging stations in rural basins used in flood-hydrograph studies.



EXPLANATION

06907080▲ U.S. GEOLOGICAL SURVEY STREAMFLOW-GAGING STATION USED IN DIMENSIONLESS HYDROGRAPH DEVELOPMENT AND REGRESSION ANALYSES

06935830△ U.S. GEOLOGICAL SURVEY STREAMFLOW-GAGING STATION USED ONLY IN REGRESSION ANALYSES

Figure 2.--Location of streamflow-gaging stations in urban basins used in flood-hydrograph studies.

Table 1.--*Basin characteristics for selected rural and urban stations*

U.S. Geological Survey station number	U.S. Geological Survey station name (figs. 1 and 2)	Contributing drainage area, A, in square miles	Impervious area, I, in percent	Channel slope, S, in feet per mile	Basin development factor, BDF	Basin lagtime, LT, in hours	Basin length, L, in miles
05495100	Big Branch tributary near Wayland[1]	0.70	3	80.8	0	1.57	1.95
05497700	Bridge Creek Branch near Baring[1]	2.54	1	43.2	0	1.92	2.25
05502700	Easdale Branch near Shelbyville[1]	0.71	1	76.1	0	1.46	1.22
05503000	Oak Dale Branch near Emden[1]	2.64	1	32.3	0	2.44	2.34
06815550	Staples Branch near Burlington Junction[1]	0.49	1	61.1	0	1.35	1.20
06816000	Mill Creek near Oregon[1]	4.90	2	42.3	0	1.62	3.30
06817980	Blacksnake Creek at St. Joseph[2]	4.32	13	38.1	1	1.38	2.80
06821000	Jenkins Branch near Gower[1]	2.72	1	34.0	0	1.32	2.55
06893558	Brush Creek at Summit Avenue at Kansas City[2]	14.4	30	27.7	11	1.21	6.31
06893570	Round Grove Creek at Raytown Road at Kansas City[2]	5.62	26	74.7	4	1.59	3.32
06893600	Rock Creek at Independence[2]	5.27	30	64.8	5	1.16	2.83
06893680	Mill Creek at 56th St. at Gladstone[2]	1.23	34	69.2	4	1.02	1.91
06896500	Thompson Branch near Albany[1]	5.58	2	30.9	0	2.73	4.75
06897700	Grand River tributary near Utica[1]	1.44	2	120	0	2.14	1.67
06902500	Hamilton Branch near New Boston[1]	2.51	1	27.0	0	2.86	3.56

Table 1.--*Basin characteristics for selected rural and urban stations--Continued*

U.S. Geological Survey station number and name (figs. 1 and 2)		Contributing drainage area, A, in square miles	Impervious area, I, in percent	Channel slope, S, in feet per mile	Basin development factor, BDF	Basin lagtime, LT, in hours	Basin length, L, in miles
06902800	Onion Branch near St. Catherine[1]	1.04	2	49.3	0	1.69	1.84
06907080	Brushy Creek tributary at Sedalia[2]	.93	25	62.4	3	.85	1.13
06910232	Flat Branch at Columbia[2]	3.01	30	49.8	6	.65	2.73
06919200	Sac River tributary near Caplinger Mills[1]	.14	1	149	0	.83	.58
06921740	Brushy Creek near Blairstown[1]	1.15	1	70.8	0	1.21	1.73
06922700	Chub Creek near Lincoln[1]	2.86	1	40.3	0	2.26	2.75
06927100	Doane Branch near Kingdom City[1]	.54	1	58.7	0	.98	1.15
06931500	Little Beaver Creek near Rolla[1]	6.41	1	65.6	0	1.74	3.45
06935800	Shotwell Creek at State Highway 340 near Ellisville[2]	.81	22	84.8	5	.65	1.10
06935830	Caulks Creek at State Highway 340 near Clarkson Valley[2]	17.1	5	33.6	3	2.63	7.71
06935880	Smith Creek at Mason Road at Creve Coeur[2]	4.44	18	53.5	4	1.88	3.21
06935890	Creve Coeur Creek at State Highway 340 near Creve Coeur[2]	22.0	15	16.4	5	4.81	8.13
06935955	Fee Fee Creek at McKelvey Road near Bridgeton[2]	11.7	25	29.4	9	2.26	4.70
06935980	Cowmire Creek at Kirchner Inc. in Bridgeton[2]	3.70	20	32.1	9	1.02	2.56

Table 1.--*Basin characteristics for selected rural and urban stations--Continued*

U.S. Geological Survey station number and name (figs. 1 and 2)		Contributing drainage area, A, in square miles	Impervious area, I, in percent	Channel slope, S, in feet per mile	Basin development factor, BDF	Basin lagtime, LT, in hours	Basin length, L, in miles
06936185	Coldwater Creek at St. Louis International Airport at Bridgeton[2]	7.47	32	30.1	9	1.46	4.65
06936380	Paddock Creek at Lindbergh Boulevard at Florissant[2]	2.64	32	29.3	9	1.00	2.56
06936460	Coldwater Creek at Old Halls Ferry Road at Florissant[2]	38.9	25	8.67	9	3.64	14.4
07002000	Watkins Creek at Coal Bank Road at St. Louis[2]	6.17	10	24.7	7	1.40	5.30
07004100	Maline Creek at Bermuda Avenue at Ferguson[2]	9.16	20	29.4	9	1.48	4.40
07005000	Maline Creek at Bellefontaine Road at Bellefontaine Neighbors[2]	24.1	25	16.4	9	2.42	8.94
07010016	River Des Peres at Hafner Place at University City[2]	5.64	25	34.4	10	1.09	4.30
07010026	River Des Peres at Pennsylvania Avenue at University City[2]	9.65	30	25.3	11	1.28	6.60
07010044	Deer Creek at Warson Road in Ladue[2]	8.59	25	29.7	9	1.34	4.25
07010061	Two Mile Creek at Trent Drive in Ladue[2]	6.42	25	32.1	9	1.20	5.24

Table 1.--*Basin characteristics for selected rural and urban stations--Continued*

U.S. Geological Survey station number and name (figs. 1 and 2)		Contributing drainage area, A, in square miles	Impervious area, I, in percent	Channel slope, S, in feet per mile	Basin development factor, BDF	Basin lagtime, LT, in hours	Basin length, L, in miles
07010086	Deer Creek at Big Bend Boulevard in Maplewood[2]	36.5	25	15.9	9	2.98	10.4
07010155	Gravois Creek at Tesson Ferry Road at Sappington[2]	12.1	32	31.1	9	1.62	6.06
07010185	Gravois Creek at Bayless Road at Bella Villa[2]	22.3	32	20.0	9	3.83	11.1
07011600	Love Branch at Rolla[2]	1.40	26	73.0	7	.65	1.68
07017115	Fox Creek at Old U.S. Highway 66 at Allenton[1]	15.6	2	41.0	0	3.34	9.25
07017500	Dry Branch near Bonne Terre[1]	3.35	1	48.5	0	1.51	4.35
07019035	Forby Creek at State Highway 109 at Eureka[1]	3.14	2	72.5	0	2.31	3.40
07019100	Fishpot Creek at Old Ballwin Road in Ballwin[2]	2.40	27	57.7	7	1.25	2.80
07019117	Fishpot Creek tributary at Sulphur Springs Road near Valley Park[2]	2.40	17	69.8	6	1.17	2.83
07019120	Fishpot Creek at Hanna Road at Valley Park[2]	9.60	25	37.0	7	1.80	7.78
07019145	Grand Glaize Creek at State Highway 141 in Manchester[2]	3.89	20	43.2	9	1.10	3.50
07019180	Grand Glaize Creek at Dougherty Ferry Road at Kirkwood[2]	19.8	22	27.2	9	3.23	6.81

Table 1.--*Basin characteristics for selected rural and urban stations--Continued*

U.S. Geological Survey station number and name (figs. 1 and 2)		Contributing drainage area, A, in square miles	Impervious area, I, in percent	Channel slope, S, in feet per mile	Basin development factor, BDF	Basin lagtime, LT, in hours	Basin length, L, in miles
07019320	Mattese Creek at Yaeger Road near Oakville[2]	9.01	25	38.8	7	1.87	5.95
07035500	Barnes Creek near Fredericktown[1]	4.03	1	114	0	2.13	4.03
07037700	Clark Creek near Piedmont[1]	4.39	1	63.9	0	2.38	3.80
07052000	Wilson Creek at Scenic Drive in Springfield[2]	19.3	22	24.4	6	2.02	7.65
07053950	Ingenthron Hollow near Forsyth[1]	.65	1	186	0	1.06	1.25
07054200	Yandell Branch near Kirbyville[1]	.33	2	116	0	1.11	1.35
07063200	Pike Creek tributary near Poplar Bluff[1]	.28	5	111	0	.82	.73
07064300	Fudge Hollow near Licking[1]	1.72	1	68.1	0	1.26	2.35
07064500	Big Creek near Yukon[1]	8.36	1	53.3	0	3.11	4.00
07068200	North Prong Little Black River near Hunter[1]	1.23	4	61.7	0	1.44	1.60

[1]Rural site.
[2]Urban site.

Several alternative approaches to hydrograph analysis and simulation were tested including methods of Commons (1942), Clark (1945), and the U.S. Department of Agriculture, Soil Conservation Service (1972). For urban streams in Missouri, the approach developed by Stricker and Sauer (1982) in a national study of urban streams is potentially applicable because their study included 25 urban sites in St. Louis County, Missouri. However, preliminary data analyses determined that the general approach of Inman (1987), if modified, would best analyze rural and urban data for Missouri. Therefore, a dimensionless hydrograph was developed for small basins in Missouri using the method given by Inman (1987). Analytical and statistical procedures utilized by Inman (1987) to develop and test dimensionless hydrographs were modified as necessary for use in the Missouri study.

The regional analysis of streamflow records provides a method for transferring the hydrologic information available at individual gaged sites to most ungaged sites within the same region where estimates might be required (Riggs, 1973). In this study, regionalization of basin lagtimes and of flood-runoff-volume data was based on multiple-regression techniques.

The relations of basin lagtimes and of flood-runoff volumes to drainage-basin characteristics were determined from regression models of the form $A=a\,B^b\,C^c\,D^d...$, where the dependent variable (A) is the basin lagtime or the flood-runoff volume and the independent variables (B, C, and D) are basin characteristics. In the equation, the regression constant is indicated by "a" and coefficients of regression are indicated by "b", "c", and "d." The regression constant and regression coefficients are defined, the statistical significance of each basin characteristic is evaluated, and a standard error of estimate is determined using regression-analysis techniques. Numerous basin and climatic characteristics were considered in this study for the regression models; however, only those of both statistical and hydrologic significance were retained in the estimating relations determined for basin lagtime and flood-runoff volume.

DIMENSIONLESS HYDROGRAPH DEVELOPED FOR MISSOURI

The dimensionless-hydrograph method used by Inman (1987) was used in the development of the dimensionless hydrograph for Missouri and subsequent statistical analyses of the data. The O'Donnell (1960) method was used to compute unit hydrographs. Preliminary results indicated that the Inman (1987) method was suitable for analysis of both rural and urban small basins in Missouri. The Inman (1987) method was considered applicable to flood hydrology in Missouri after preliminary analysis of data for about 15 gaged sites. However, to avoid geographic bias, to show statewide applicability to both rural and urban basins, and to develop a dimensionless hydrograph specific to Missouri, 341 flood hydrographs from 41 gaged sites in Missouri were eventually analyzed. It was concluded that the dimensionless hydrograph was adequately defined based on data from these 41 gaged sites.

To develop a dimensionless hydrograph applicable to small basins in Missouri, data for 24 rural and 17 urban gaging stations (figs. 1 and 2) were analyzed in detail. For these analyses, the basin lagtime was computed as the time at the centroid of the unit hydrograph minus one-half the time of the computation interval (duration). Actual flood hydrographs (for example, fig. 3) were analyzed to obtain a unit hydrograph of given duration and the basin lagtime for each flood for each site (average of about eight floods per site). Then an average unit hydrograph and an average basin lagtime were computed for each site. The process of averaging unit hydrographs is presented in table 2.

These average unit hydrographs were transformed (Inman, 1987, p. 3) to unit hydrographs having generalized durations of one-fourth, one-third, one-half, and three-fourths of the average basin lagtime for each gaged site. Dimensionless hydrographs were obtained by dividing the time by basin lagtime and the discharge by peak discharge. The four generalized-duration dimensionless hydrographs, average basin lagtimes, and peak discharges from the actual flood hydrographs were used to simulate flood hydrographs. Widths of the simulated flood hydrographs, from the four generalized-duration dimensionless hydrographs, were compared with widths of the corresponding actual flood hydrographs at 50- and 75-percent of peak discharge. Based on analyses of the data for Missouri, the unit hydrographs of one-half the average basin lagtime duration best fit the recorded data. This was expected based on the experience of Inman (1987, p. 5). The range of the station data and the average dimensionless hydrograph of one-half the average basin lagtime duration are shown in figure 4.

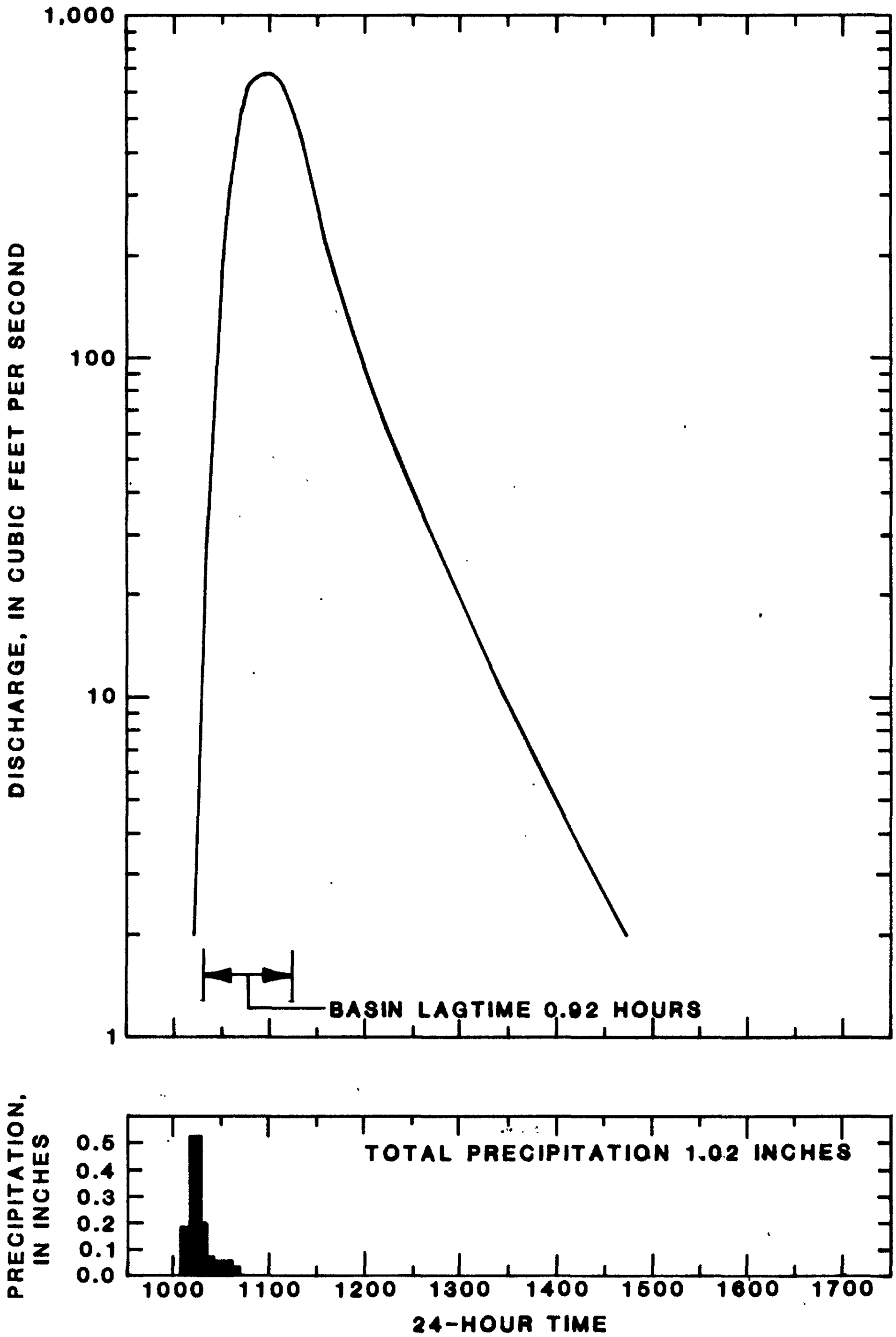


Figure 3.--Actual flood hydrograph and unit precipitation at Cowmire Creek at Kirchner Inc., in Bridgeton, Missouri (06935980) for flood of August 23, 1977.

Table 2.--*Discharge at 15-minute intervals for seven unit hydrographs and the average unit hydrograph computed for Chub Creek near Lincoln, Missouri (06922700)*

Discharge, in cubic feet per second							
Unit hydrograph for indicated date							Average
April 14, 1965	July 1, 1965	September 5, 1965	June 26, 1967	June 22, 1969	July 2, 1969	July 3, 1969	unit hydrograph
0	0	0	0	0	0	0	0
0	0	0	62	0	0	0	9
0	0	0	116	0	0	0	17
0	44	0	299	14	208	0	81
126	374	0	432	35	383	110	209
656	593	169	477	158	555	339	421
931	676	397	540	394	582	550	581
1,020	734	595	601	592	572	689	686
1,100	756	641	623	708	602	739	739
1,040	739	629	606	698	543	674	704
952	685	629	569	616	439	531	632
823	626	621	525	530	371	403	558
672	560	572	489	485	311	295	483
497	487	530	447	468	300	233	423
352	402	501	392	449	259	227	369
225	324	457	328	425	205	244	316
138	233	420	262	378	216	273	274
63	135	382	199	343	210	284	231
27	101	308	150	306	184	266	192
17	68	225	115	246	177	224	153
9	25	182	84	215	148	172	119
7	9	160	52	159	146	124	94
4	4	121	25	124	148	88	73
3	3	81	12	111	118	77	58
2	1	62	7	77	109	87	49
2	1	41	4	54	99	103	43
1	0	37	3	28	80	113	37
1	0	58	1	2	78	117	37
0	0	56	0	0	55	111	30
0	0	22	0	0	42	87	22
0	0	0	0	0	50	52	15
0	0	0	0	0	38	32	10
0	0	0	0	0	33	27	9
0	0	0	0	0	33	22	8
0	0	0	0	0	25	25	7
0	0	0	0	0	29	42	10
0	0	0	0	0	26	59	12
0	0	0	0	0	16	58	11
0	0	0	0	0	16	47	9
0	0	0	0	0	10	36	7
0	0	0	0	0	4	18	3
0	0	0	0	0	2	0	0
0	0	0	0	0	0	0	0

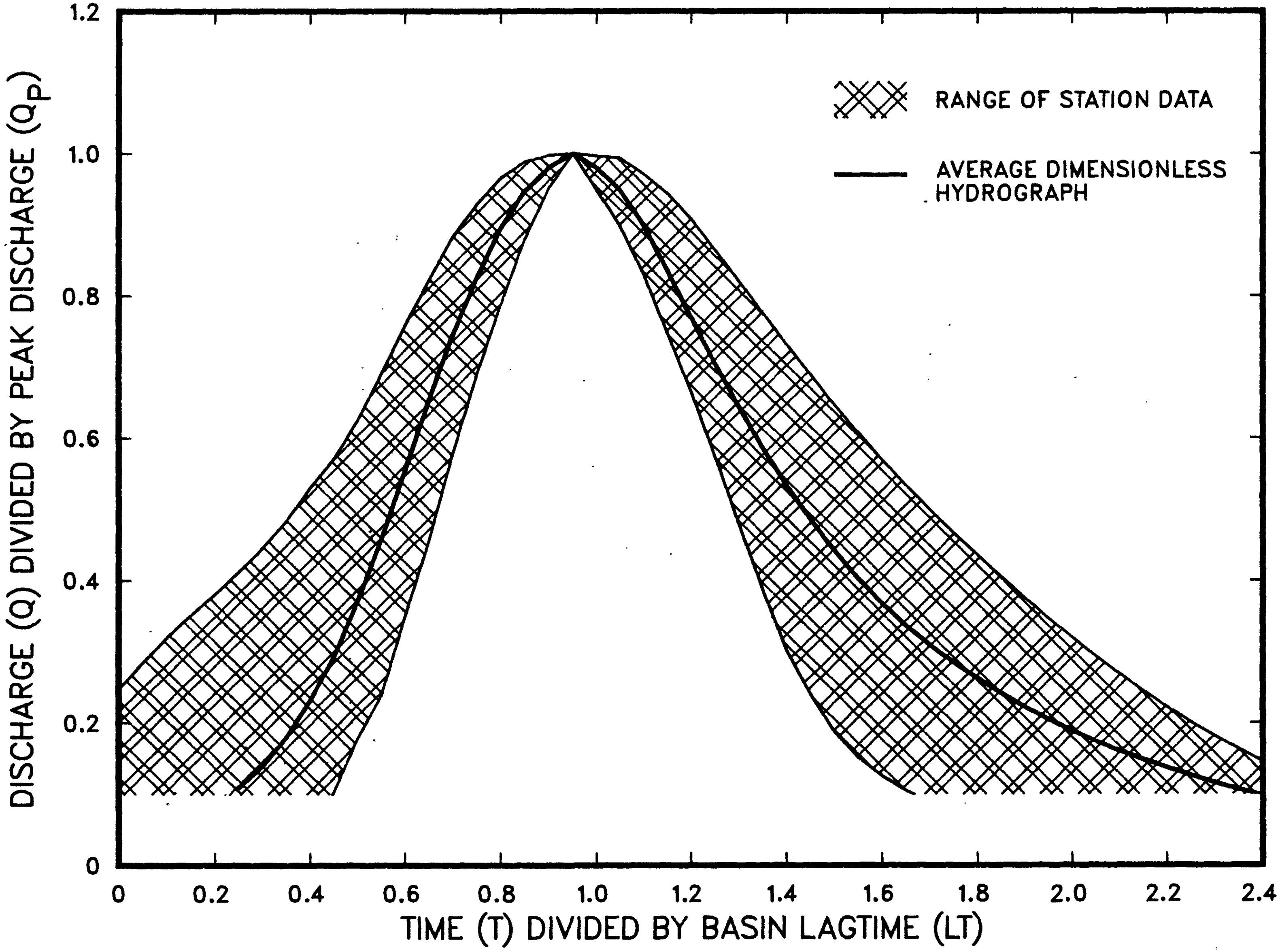


Figure 4.-- Range of data from 41 stations and average dimensionless hydrograph of one-half the basin lagtime duration.

This average dimensionless hydrograph, applicable to both rural and urban basins in Missouri, was obtained by combining dimensionless hydrographs determined for each of the 41 gaged sites. Coordinates (time and discharge ratios) of the dimensionless hydrograph for Missouri are given in table 3 and are plotted in figure 5.

The dimensionless hydrograph for Missouri (fig. 5) was used with the average basin lagtimes and peak discharges from the actual flood hydrographs to simulate flood hydrographs for comparison with the actual flood hydrographs. The widths of simulated flood hydrographs and actual, single-peak flood hydrographs were again compared at 50- and 75-percent of their peak discharges. At the width of the 50-percent peak discharge, the standard error of estimate was ±37.8 percent; at the width of the 75-percent of peak discharge, the standard error of estimate was ±42.6 percent.

The dimensionless hydrograph developed for Missouri closely approximates the dimensionless hydrograph developed for Georgia (Inman, 1987), which also was verified in central Tennessee (Robins, 1986) and other areas (Sauer, in press). A comparison of the two dimensionless hydrographs is shown in figure 6.

Based on standard errors of estimate, the simulated flood hydrographs obtained using the dimensionless hydrograph developed for Missouri more closely matched the data recorded in Missouri than did the simulated flood hydrographs obtained using the dimensionless hydrograph developed for Georgia. The dimensionless hydrograph developed for Georgia is 4.4 percent wider at 50-percent and 7.3 percent wider at 75-percent of the Q/Q_p ratio widths than the one developed for Missouri. Also, the summations of the discharge ordinates for the dimensionless hydrographs (fig. 6) differ by 4.0 percent. The minor differences between the two dimensionless hydrographs are the result of the different hydrologic settings of basins in Missouri and Georgia.

As noted earlier, hydrograph-simulation methods investigated for possible use included those of the U.S. Department of Agriculture, Soil Conservation Service (1972) and of Stricker and Sauer (1982). Their dimensionless hydrographs are compared to the dimensionless hydrograph developed for Missouri in figure 7. Because the Soil Conservation Service dimensionless hydrograph was derived using the time to peak (Tp) rather than the basin lagtime (LT), it is only indirectly comparable to the other dimensionless hydrographs in figure 7.

ESTIMATES OF BASIN LAGTIME AND PEAK DISCHARGE

Simulation of flood hydrographs for small basins in Missouri requires estimates of basin lagtime and peak discharge as well as use of the dimensionless hydrograph discussed previously. Because of the need for these estimates in simulating flood hydrographs, methods for estimating basin lagtime and peak discharge are presented in this section.

Estimating Basin Lagtime

The dimensionless hydrograph is based on drainage-basin response time, commonly referred to as basin lagtime. As noted by Stricker and Sauer (1982), basin lagtime generally is considered constant for a basin and is defined as time between the time of the centroid of rainfall excess and the time of the centroid of the runoff hydrograph. This time characteristic of a basin (lagtime) is a principal factor determining the relative shape of runoff hydrographs.

For gaged basins, basin lagtime can be determined by analyzing the timing of rainfall and resultant runoff from individual storms for each basin, and averaging these results to obtain an average basin lagtime. However, for ungaged basins, estimates of basin lagtime need be made, so estimating relations based on other basin characteristics were developed. Basin characteristics used in multiple regressions (See "Analytical Procedures") to determine basin lagtime included drainage area (A), impervious area (I), basin development factor (BDF), basin length (L), and main-channel slope (S).

Equations for estimating peak discharges of given frequency are provided by Becker (1986), based on a previous investigation of small streams in Missouri. These equations are based on the basin characteristics of drainage area (A), impervious area (I), and basin development factor (BDF). Therefore, it is desirable that equations for estimating basin lagtime also be based on these same basin characteristics.

Table 3.--*Time and discharge ratios of dimensionless hydrograph developed for Missouri*

[T, time, in hours; LT, basin lagtime, in hours; Q, discharge, in cubic feet per second; Q_p, peak discharge, in cubic feet per second]

Time ratio (T/LT)	Discharge ratio (Q/Q_p)
0.25	0.11
.30	.14
.35	.18
.40	.23
.45	.29
.50	.37
.55	.46
.60	.55
.65	.65
.70	.74
.75	.83
.80	.89
.85	.95
.90	.98
.95	1.00
1.00	.98
1.05	.95
1.10	.90
1.15	.84
1.20	.77
1.25	.71
1.30	.65
1.35	.59
1.40	.53
1.45	.48
1.50	.44
1.55	.40
1.60	.37
1.65	.34
1.70	.31
1.75	.28
1.80	.26
1.85	.24
1.90	.22
1.95	.20
2.00	.19
2.05	.17
2.10	.16
2.15	.15
2.20	.14
2.25	.13
2.30	.12
2.35	.11
2.40	.10

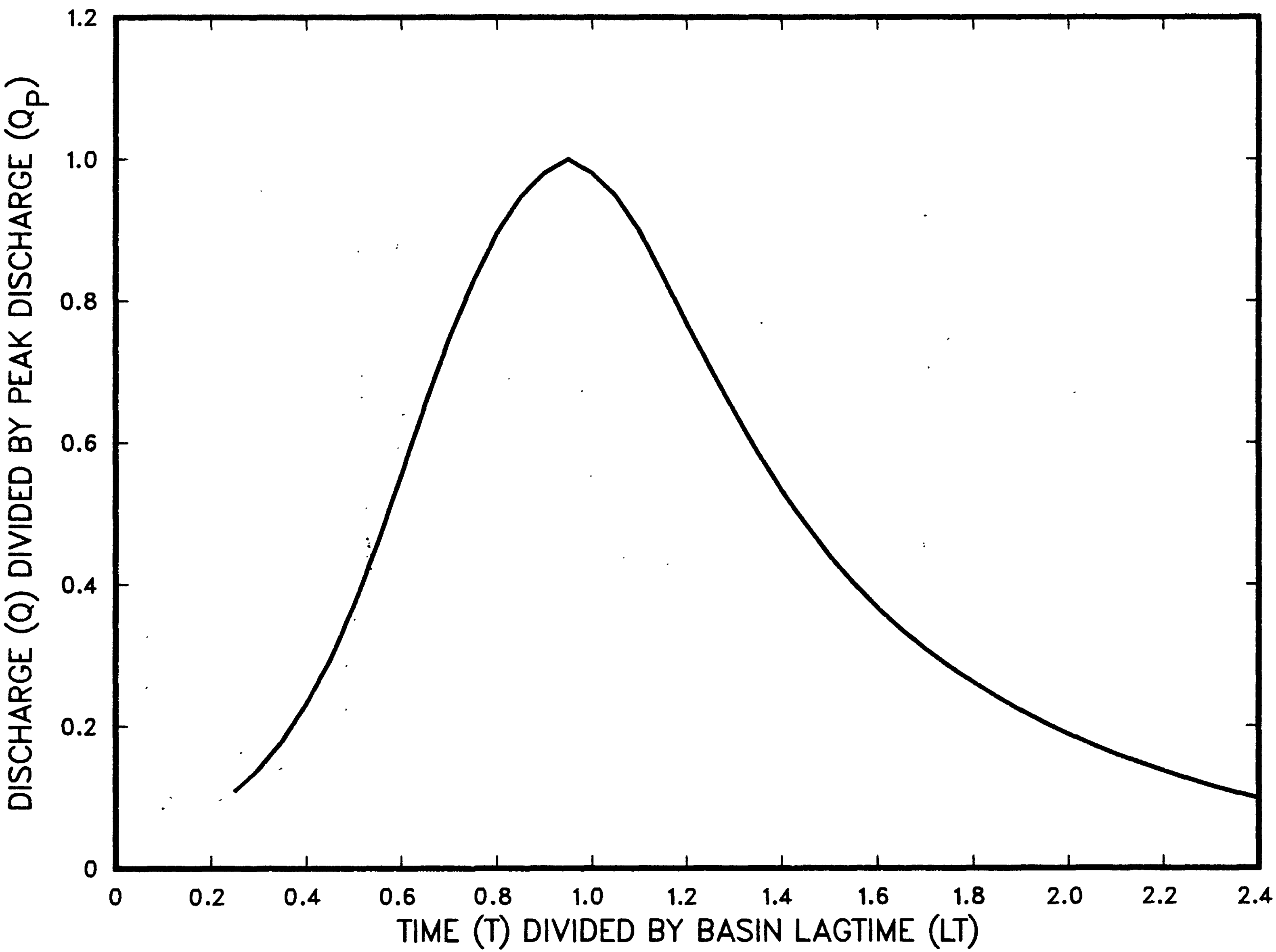


Figure 5.—— Dimensionless hydrograph developed for small basins in Missouri.

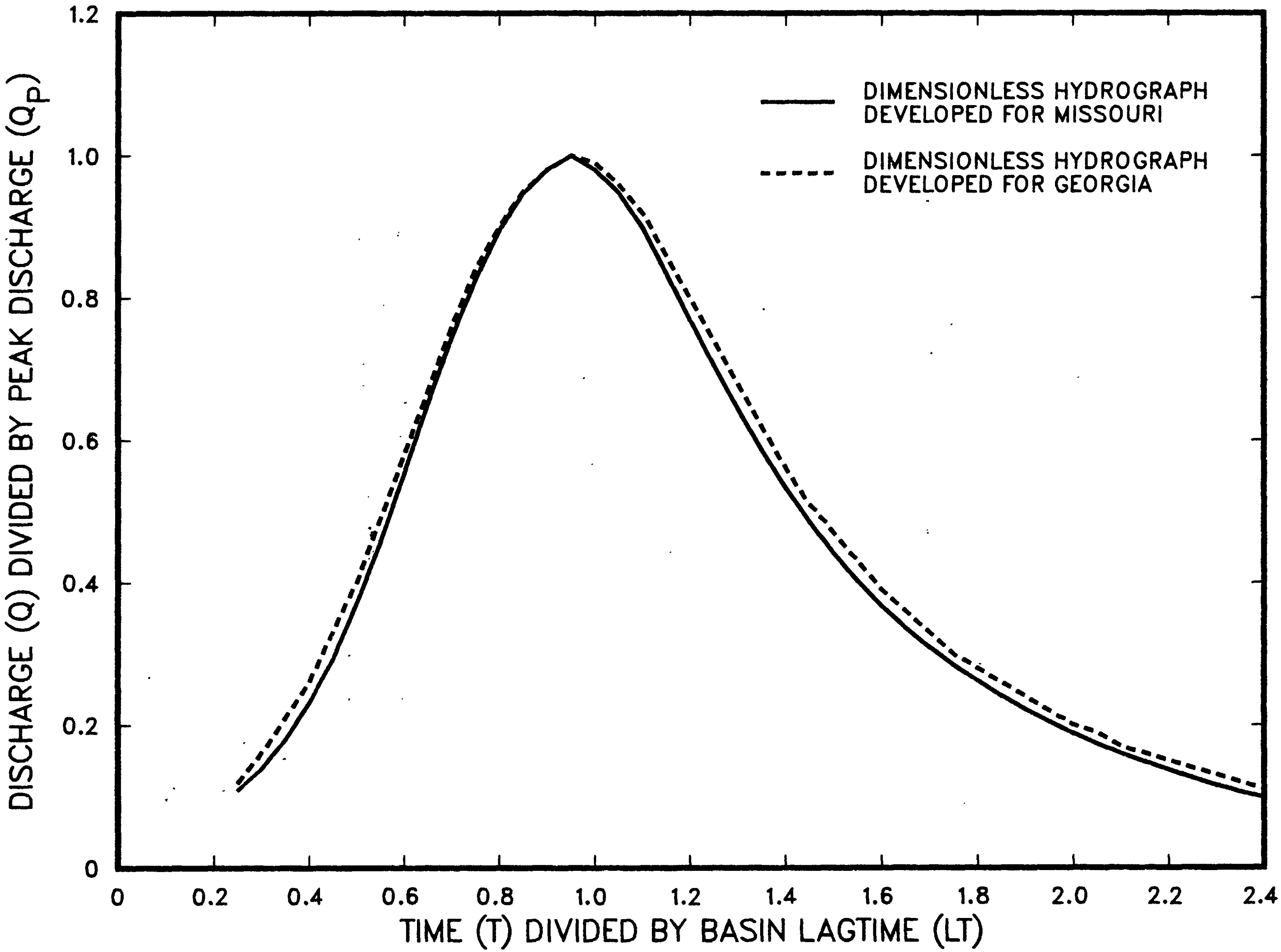


Figure 6.—— Comparison of dimensionless hydrograph developed for Missouri with dimensionless hydrograph developed for Georgia (Inman, 1987).

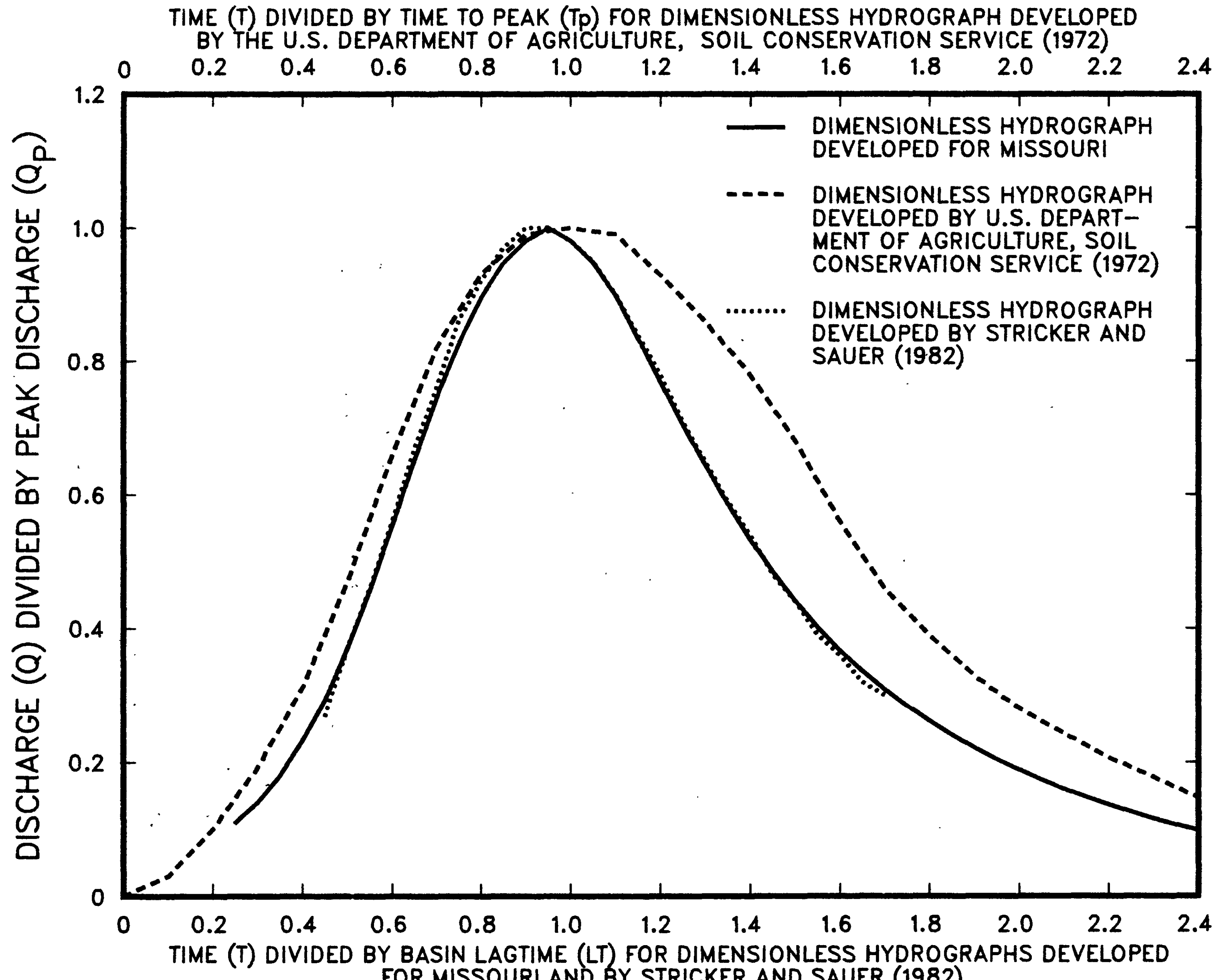


Figure 7.—— Comparison of dimensionless hydrograph developed for Missouri with dimensionless hydrographs developed by the U.S. Department of Agriculture, Soil Conservation Service (1972) and Stricker and Sauer (1982).

In a nationwide study of basin lagtime, Sauer and others (1983) included 25 urban stations located in St. Louis County, Missouri. Basin characteristics determined to be significant in that study were tested for significance in the statewide study of basin lagtime in Missouri. Basin characteristics for the 25 urban stations in St. Louis County and 9 urban stations located elsewhere in Missouri (fig. 2) were used in a regression of LT versus $L/\sqrt{S}$ and BDF. As expected, these basin characteristics were useful in determining basin lagtime for ungaged urban basins in Missouri.

Final regression equations utilizing these basin characteristics for estimation of basin lagtime are based on combining the characteristics for the 27 rural and 34 urban basins listed in table 1. Equations for estimating basin lagtime for combined rural and urban basins, and urban basins only, are listed in table 4.

Table 4.--*Summary of equations for estimating basin lagtime developed for small basins in Missouri*

[LT, basin lagtime, in hours; A, drainage area, in square miles; I, impervious area, in percent; BDF, basin development factor; L, basin length, in miles; S, main-channel slope, in feet per mile]

Equation number	Equation	Equation applicability	Standard error of estimate (percent)
(1)	$LT=1.46\ A^{0.34}\ I^{-0.19}$	Rural and urban basins	±26.3
(2)	$LT=0.34\ A^{0.37}(13\text{-BDF})^{0.52}$	Rural and urban basins	±27.0
(3)	$LT=0.86\ (L/\sqrt{S})^{0.60}(13\text{-BDF})^{0.45}$	Urban basins	±23.2

Based on data for the 34 urban basins in Missouri, equation 3, $LT = 0.86\ (L/\sqrt{S})^{0.60}\ (13\text{-BDF})^{0.45}$, was obtained (see table 4). This may be compared with the equation $LT = 0.85\ (L/\sqrt{S})^{0.62}\ (13\text{-BDF})^{0.47}$ determined by Sauer and others (1983).

Accuracy of the equations in table 4 are indicated by the average standard error of estimate. Regression residuals were compared for these equations to evaluate possible bias when rural and urban basins were combined. Equations were not significantly biased geographically, nor were they significantly biased because of combining rural and urban basins.

Estimating Peak Discharge

For small basins in Missouri, flood data for gaged sites are given in Hauth, 1974b; Spencer and Alexander, 1978; and Becker, 1986. Peak discharges at ungaged rural and urban sites can be estimated using one of two sets of regression equations (Becker, 1986) relating flood magnitude to basin characteristics. Forms of the equations are:

$$Q_t = a\,A^b\,I^c \qquad (4)$$

and

$$Q_t = d\,A^e\,BDF^f \qquad (5)$$

where
- Q = peak discharge, in cubic feet per second;
- t = recurrence interval, in years;
- a and d = regression constants;
- b, c, e, and f = regression coefficients;
- A = contributing drainage area, in square miles;
- I = impervious area, in percent; and
- BDF = basin development factor.

Alternative peak-discharge solutions, of comparable accuracy, (equations 4 and 5) provide planners a choice of methods for estimating peak discharge in rural and urban basins. Depending on basin type and location, it may be easier to determine a basin development factor (BDF) than to determine the percentage of impervious area (I) or, conversely, the opposite may be the case. For convenience, equations for estimating the peak discharge of floods having a 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval (Becker, 1986) are tabulated in the "Supplemental Data" section at the back of this report.

For small basins, the equations for estimating peak discharge that are presented in Becker (1986) are considered applicable to both rural and urban ungaged sites because the analyses included data from both rural and urban gaged sites. In that study, rural basins were included in the regional analysis of urban basins to extend the gaged-data sample in areal coverage and to extend the applicability of equations developed. It is reasonable to consider a rural site as representing an urban site wherein the effects of urbanization are nonexistent or virtually zero. However, most rural basins will have some effective impervious area. Therefore, a small percentage of impervious area, based on roads, ponds, and so forth, was determined or assumed (minimum of 1 percent) for each rural basin used in the regression analyses. Alternative selections of rural sites were tested in the regionalization process to assure that comparable equations would be obtained and that the data were not biased.

The reliability of peak-discharge estimates is indirectly indicated by the standard errors of estimate (See "Supplemental Data") of the regression equations. The difference between the estimated and the actual peak discharge for two-thirds of the estimates will be within plus or minus one standard error of estimate. The probability of one or more floods exceeding a flood of given recurrence interval (the t-year flood) within a given period of years can be estimated. Procedures for making these risk estimates are given by the U.S. Water Resources Council (1981).

TECHNIQUE FOR SIMULATING FLOOD HYDROGRAPHS

A flood hydrograph for small basins in Missouri, both rural and urban, can be simulated from the time and discharge ratios of the dimensionless hydrograph developed for Missouri (table 3). The expansion of this dimensionless hydrograph is accomplished by multiplying each abscissa value (T/LT) by LT and each ordinate value (Q/Q_p) by Q_p, where LT is the estimated basin lagtime for the drainage basin and Q_p is the flood-peak discharge. The resulting simulated flood hydrograph has a peak-discharge value equal to the flood-peak-discharge (Q_p) value. Because the dimensionless hydrograph is defined between the time ratios (T/LT) of 0.25 and 2.40, the simulated flood hydrograph has a time base, in hours, equal to the basin lagtime (LT), in hours, multiplied by 2.15.

The validity of using the dimensionless hydrograph developed for Missouri to simulate flood hydrographs was tested in several ways. After the dimensionless hydrograph was developed, hydrographs were simulated for 341 floods using the dimensionless hydrograph, recorded peak discharge, and average basin lagtime computed from recorded data. Simulated and actual hydrographs for all floods considered were plotted for comparison.

The simulated flood hydrographs in many instances nearly duplicated the actual flood hydrographs. Comparisons of simulated and actual flood hydrographs for selected floods in selected rural and urban gaged basins are shown in figures 8 and 9. Obviously, some flood hydrographs will not be simulated as closely as those shown in figures 8 and 9. Complex (multiple peak) flood hydrographs do not compare well with flood hydrographs simulated using the single-peak dimensionless hydrograph developed for Missouri, as shown in figure 10. Further, the dimensionless hydrographs developed for each of the 41 stations varied somewhat from the average dimensionless hydrograph as shown in figure 4. However, use of the dimensionless hydrograph can produce simulated flood hydrographs that closely approximate actual, single-peak flood hydrographs for both rural and urban basins in Missouri.

A statistical check of the closeness of the fit of the simulated flood hydrographs to the actual flood hydrographs was made. This involved comparing hydrograph widths at 50- and 75-percent of the peak discharge (Q_p) for the simulated and actual flood hydrographs (see fig. 10). Examples of the comparisons involved in making this statistical check of the hydrograph fit are listed in table 5 for the flood hydrographs shown in figures 8 and 9. The closeness of fit was judged by the average difference in widths (percent) for all single-peak flood hydrographs. For 273 recorded single-peak floods considered in this comparison, the standard error of estimate was ±37.8 percent for the 50-percent peak-discharge width and ±42.6 percent for the 75-percent peak-discharge width.

HYDROGRAPH-WIDTH RELATION

A complete flood hydrograph might not be required for all design analyses. For example, only the period of time that a specified discharge is exceeded by a flood of a given recurrence interval might be needed to evaluate risks associated with a design analysis. Therefore, a hydrograph-width relation is defined from the dimensionless hydrograph developed for Missouri. This relation is shown in figure 11 and the ratios from which the relation is defined are given in table 6.

The time that discharge exceeds a specified value can be represented by the width (W) of the flood hydrograph at the specified value. A hydrograph-width ratio (W/LT) was determined by subtracting the value of T/LT on the rising limb from the value of T/LT on the falling limb of the dimensionless hydrograph (fig. 5), at the same Q/Q_p discharge ratio. These hydrograph-width ratios (W/LT) were plotted in relation to the discharge ratios (Q/Q_p). Hydrograph width (W), in hours, is determined by multiplying the appropriate hydrograph-width ratio (W/LT), for the desired discharge ratio (Q/Q_p), by basin lagtime (LT). See "Application Examples", example 2.

FLOOD-RUNOFF-VOLUME RELATION

During investigations of hydrograph shape, computations of the flood-runoff volumes associated with the recorded flood peaks were made for 193 single-peak floods. These flood-runoff volumes were regressed with peak discharges and selected drainage-basin characteristics (see "Analytical Procedures") to obtain an equation for estimating flood-runoff volume. In general, 5 of the larger, single peak, floods recorded at each of 24 rural and 17urban gaging stations were used to obtain an unbiased sampling.

As an alternative, an estimate of flood-runoff volume can be computed from the simulated flood hydrograph obtained using the peak discharge and estimated basin lagtime. The estimate of flood-runoff volume is calculated by integrating the discharge, in cubic feet per second, over the time base of the hydrograph and converting to runoff, in acre-feet.

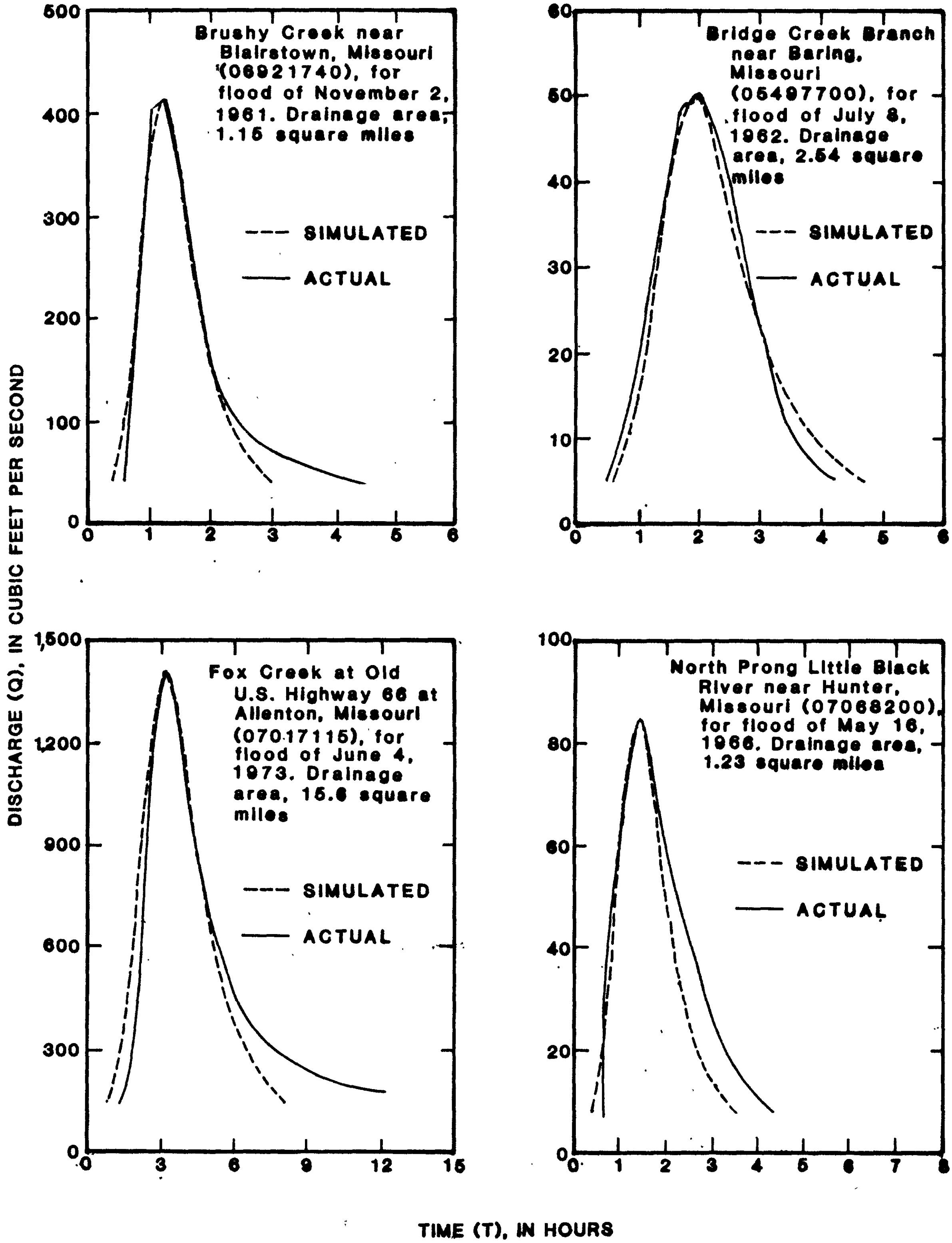


Figure 8.--Comparison of simulated and actual flood hydrographs for selected rural basins in Missouri.

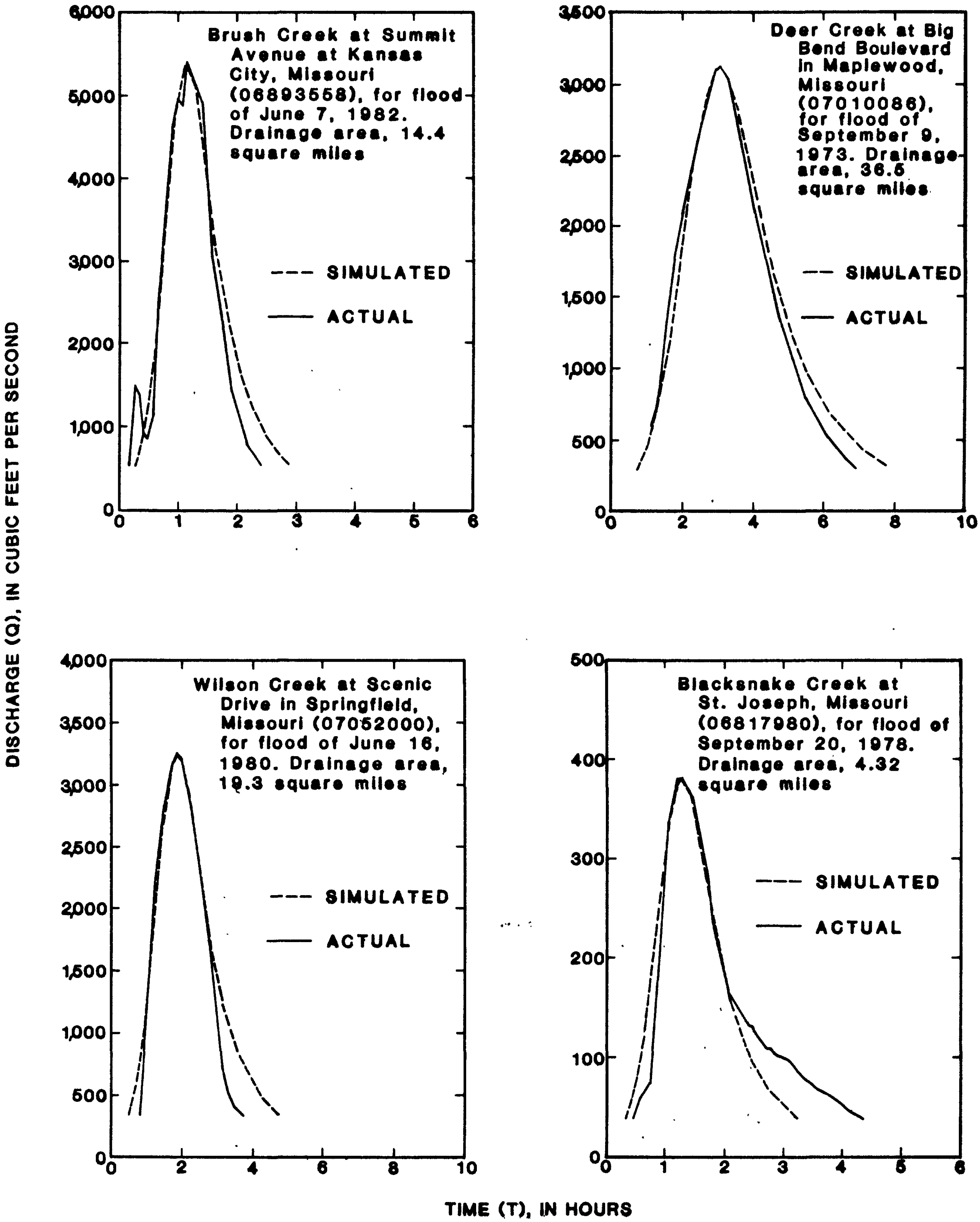


Figure 9.--Comparison of simulated and actual flood hydrographs for selected urban basins in Missouri.

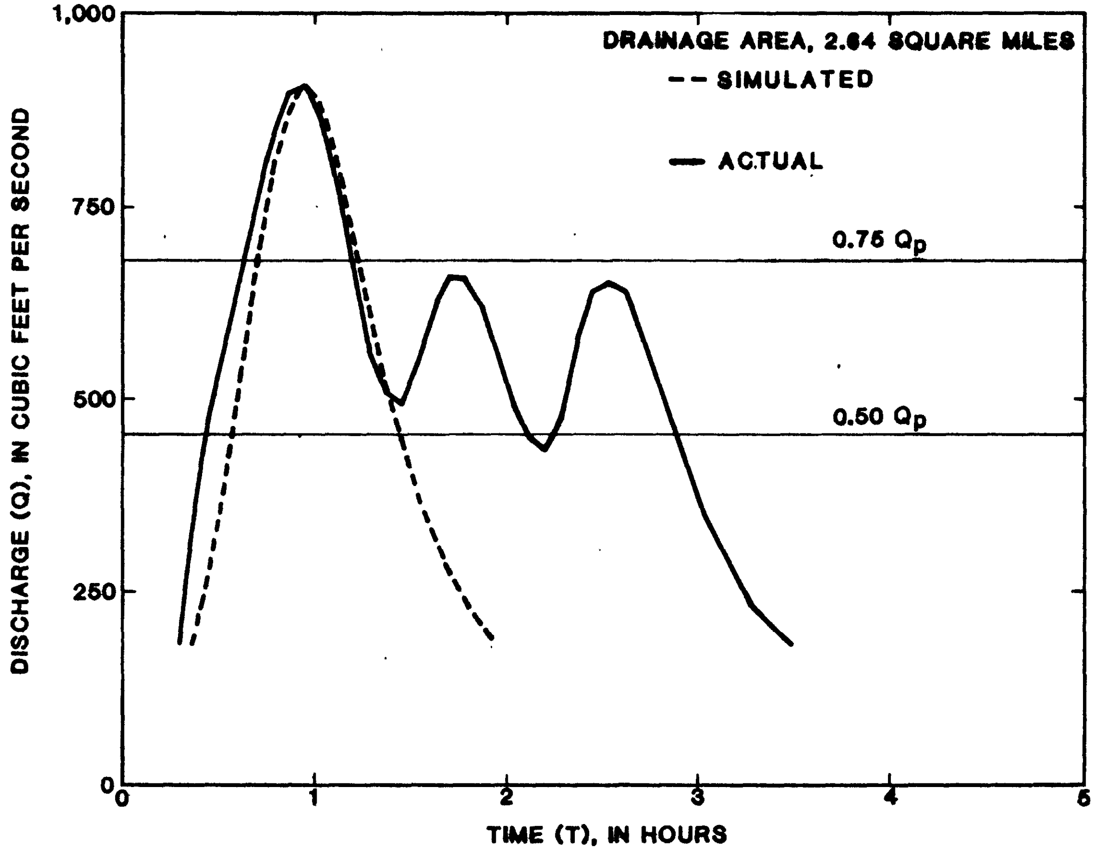


Figure 10.--Comparison of simulated, single-peak flood hydrograph and actual, multiple-peak flood hydrograph, Paddock Creek at Lindbergh Boulevard at Florissant, Missouri (06936380) for flood of August 28, 1974.

Table 5.--*Hydrograph-width comparisons for selected stations*

U.S. Geological Survey station number and name (figs. 1, 2, 8, and 9)		Date of flood	Peak discharge (cubic feet per second)	Width at 50 percent of peak discharge: Simulated hydrograph (hours)	Width at 50 percent of peak discharge: Actual hydrograph (hours)	Width at 50 percent of peak discharge: Difference (percent)	Width at 75 percent of peak discharge: Simulated hydrograph (hours)	Width at 75 percent of peak discharge: Actual hydrograph (hours)	Width at 75 percent of peak discharge: Difference (percent)
05497700	Bridge Creek Branch near Baring[1]	7-08-62	50	1.67	1.77	-5.6	1.00	1.11	-9.9
06817980	Blacksnake Creek at St. Joseph[2]	9-20-78	382	1.19	1.11	+7.2	.72	.74	-2.7
06893558	Brush Creek at Summit Avenue at Kansas City[2]	6-07-82	5,410	1.03	.99	+4.0	.62	.66	-6.1
06921740	Brushy Creek near Blairstown[1]	11-02-61	413	1.05	1.04	+1.0	.62	.65	-4.6
07010086	Deer Creek at Big Bend Boulevard in Maplewood[2]	9-09-73	3,130	2.84	2.83	+ .4	1.70	1.60	+6.2
07017115	Fox Creek at Old U.S. Highway 66 at Allenton[1]	6-04-73	1,400	2.88	2.74	+5.1	1.73	1.63	+6.1
07052000	Wilson Creek at Scenic Drive in Springfield[2]	6-16-80	3,250	1.75	1.69	+3.6	1.05	1.10	-4.5
07068200	North Prong Little Black River near Hunter[1]	5-16-66	85	1.25	1.73	-27.8	.75	.85	-11.8

[1]Rural site.
[2]Urban site.

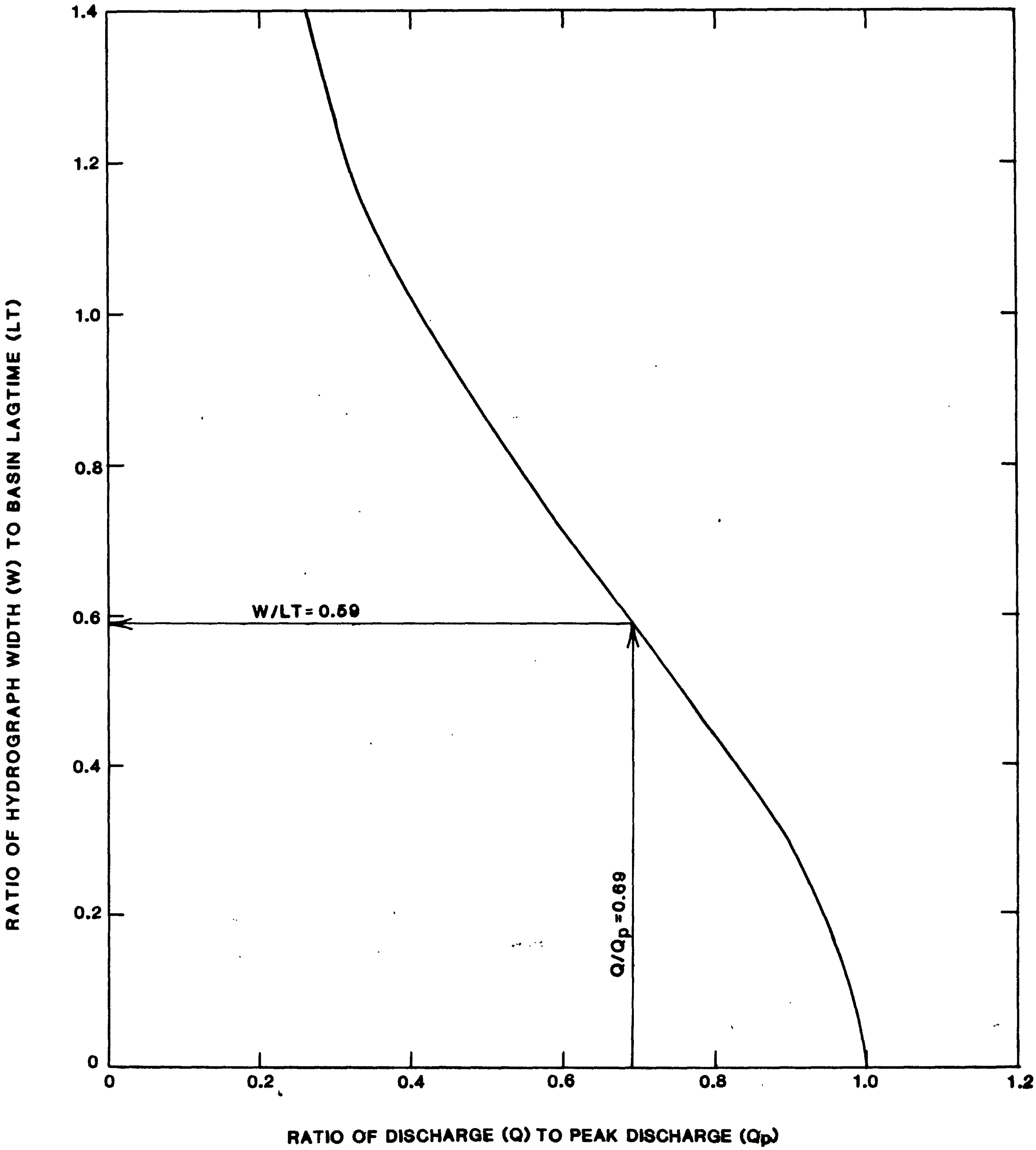


Figure 11.--Hydrograph-width relation for dimensionless hydrograph developed for Missouri as used in application example 2.

Table 6.--*Hydrograph-width and discharge ratios for dimensionless hydrograph developed for Missouri*

[W, hydrograph width, in hours; LT, basin lagtime, in hours; Q, discharge, in cubic feet per second; Q_p, peak discharge, in cubic feet per second]

Hydrograph-width ratio (W/LT)	Discharge ratio (Q/Q_p)
0	1.00
.19	.95
.29	.90
.37	.85
.44	.80.
.51	.75
.58	.70
.65	.65
.71	.60
.79	.55
.86	.50
.94	.45
1.03	.40
1.14	.35
1.26	.30
1.41	.25
1.59	.20

Estimating Flood-Runoff Volume

Peak discharge and lagtime were significant as independent variables in deriving the following regression equation (standard error of estimate = ±32.3 percent) for flood-runoff volume:

$$V = 0.0702\, Q_p^{1.035}\, LT^{0.913} \qquad (6)$$

where V = flood-runoff volume, in acre-feet,
Q_p = peak discharge, in cubic feet per second, and
LT = basin lagtime, in hours.

The above equation may be useful where storage (flood-runoff volume) is a design consideration or is used in risk analysis. An estimate of flood-runoff volume associated with a peak discharge of given frequency, such as a 50- or 100-year recurrence interval, can be made using equation 6.

The alternative integration process indicated previously also can be applied to the dimensionless hydrograph developed for Missouri (fig. 5 and table 3). For this calculation, the rising and falling limbs of the dimensionless hydrograph are extrapolated to zero discharge. A numerical integration, by approximation using rectangular areas, of the extrapolated dimensionless hydrograph provides a dimensionless result. This dimensionless result is then multiplied by Q_p and LT because of the need to expand the dimensionless hydrograph. Conversion of basin lagtime, in hours, to seconds and volume, in cubic feet, to acre-feet results in an equation for estimating flood-runoff volume:

$$V = 0.085\, Q_p\, LT \qquad (7)$$

where V = flood-runoff volume, in acre-feet,
Q_p = peak discharge, in cubic feet per second, and
LT = basin lagtime, in hours.

The above analyses provides alternative methods for estimating flood-runoff volumes that provide estimates of about equal accuracy as shown below.

Comparison of Actual and Estimated Flood-Runoff Volumes

Statistical comparisons were needed to verify that estimating procedures, based on regression and numerical-integration methods, would provide reasonable accuracy. Runoff volumes for 193 single-peak floods were computed using actual flood hydrographs. Estimates of flood-runoff volume were computed for each of these hydrographs using equations 6 and 7. Differences between actual and both estimated flood-runoff volumes were computed, and the percentage differences between these were calculated. The averaged percentage differences were used in comparing the overall accuracy of the two estimating procedures. On the average, it was determined that equation 7 estimated the actual flood-runoff volume by about 0.6 percent greater than did equation 6 for the 193 floods at the 41 stations. Comparisons of flood-runoff-volume values are presented in table 7 for the selected flood hydrographs shown in figures 8 and 9.

LIMITATIONS

The approach of this study was to analyze flood-hydrograph data for rural and urban basins with drainage areas of less than about 40 square miles. Limitations of technique and equations, herein, are based on a general requirement for equivalence of the ungaged site and the range of the data sample used in the analyses leading to development of the technique and equations. Basin characteristics for the data sample ranged as follows:

Basin characteristic	Range of data
Contributing drainage area	0.28 to 38.9 square miles
Basin development factor	0 to 11
Impervious area	1 to 34 percent
Basin lagtime	0.65 to 4.81 hours
Basin length	0.58 to 14.4 miles
Main-channel slope	8.7 to 186 feet per mile

Therefore, the technique for simulating flood hydrographs and the equations for estimating basin lagtime and flood-runoff volume might not provide reliable results for sites where basin characteristics have values smaller or larger than the sampled range. The technique and equations are applicable only to sites where flood flows are relatively unaffected by storage or diversions; therefore, they are not applicable where major dams or intrabasin diversions substantially affect peak discharge. The applicability of hydrograph-simulation technique and equations needs to be judged by the possible effect expected on hydrograph magnitude and shape caused by such features.

APPLICATION EXAMPLES

The following examples are given to illustrate the use of the technique and equations provided in this report.

Example 1.--Simulate the flood hydrograph and estimate the flood-runoff volume corresponding to a 100-year flood-peak discharge on an ungaged small basin in a city where the effects of urbanization are great. Assume that the contributing drainage area (A) is 5.00 square miles and that detailed mapping or an onsite reconnaissance has determined that an appropriate value for the basin development factor (BDF) is 8. Estimates of basin lagtime (LT) and of peak discharge (Q_p) need to be made before a flood hydrograph, corresponding to the 100-year flood, can be simulated.

Table 7.--*Flood-runoff-volume comparisons for selected basins*

U.S. Geological Survey station number and name (figs. 1, 2, 8, and 9)		Date of flood	Peak[1] discharge, Q_p (cubic feet per second)	Basin lagtime, LT (hours)	Actual[1] flood-runoff volume (acre-feet)	Estimated flood-runoff volume using equation 6 (acre-feet)	Percentage difference from actual flood-runoff volume	Estimated flood-runoff volume using equation 7 (acre-feet)	Percentage difference from actual flood-runoff volume
05497700	Bridge Creek Branch near Baring[2]	7-08-62	47	1.92	6.98	6.85	-1.9	7.67	+9.9
06817980	Blacksnake Creek at St. Joseph[3]	9-20-78	352	1.38	39.4	40.7	+3.3	41.3	+4.8
06893558	Brush Creek at Summit Avenue at Kansas City[3]	6-07-82	5,350	1.21	505	604	+19.5	550	+8.9
06921740	Brushy Creek near Blairstown[2]	11-02-61	407	1.21	44.8	42.0	-6.4	41.9	-6.6
07010086	Deer Creek at Big Bend Boulevard in Maplewood[3]	9-09-73	3,070	2.98	766	774	+1.0	778	+1.5
07017115	Fox Creek at Old U.S. Highway 66 at Allenton[2]	6-04-73	1,310	3.34	344	356	+3.5	372	+8.1
07052000	Wilson Creek at Scenic Drive in Springfield[3]	6-16-80	3,110	2.02	448	550	+22.8	534	+19.2
07068200	North Prong Little Black River near Hunter[2]	5-16-66	85	1.44	13.1	9.72	-25.8	10.4	-20.6

[1]Adjusted for base flow.
[2]Rural site.
[3]Urban site.

Solution:

(1) Basin lagtime (LT) of 1.42 hours is estimated by substitution in equation 2 (table 4) when A = 5.00 and BDF = 8.

$$LT = 0.34\ A^{0.37}\ (13\text{-}BDF)^{0.52}$$
$$LT = 0.34\ (5.00)^{0.37}\ (13\text{-}8)^{0.52} = 1.42 \text{ hours}$$

(2) Equation 19, in the Supplemental Data section, provides a peak-discharge estimate for the 100-year flood of 5,850 cubic feet per second when A = 5.00 and BDF = 8.

$$Q_{100} = 2{,}820\ A^{0.783}\ (13\text{-}BDF)^{-0.330}$$
$$Q_{100} = 2{,}820\ (5.00)^{0.783}\ (13\text{-}8)^{-0.330} = 5{,}850$$
$$Q_p = Q_{100} = 5{,}850 \text{ cubic feet per second}$$

(3) Compute time (T) and discharge (Q) for coordinates of the simulated flood hydrograph where basin lagtime (LT) is 1.42 hours and peak discharge (Q_p) is 5,850 cubic feet per second. The computation of coordinates for the simulated flood hydrograph is presented in table 8, and the simulated flood hydrograph is shown in figure 12.

(4) The flood-runoff volume (V) can be estimated, based on the numerical-integration method, by using equation 7.

$$V = 0.085\ Q_p\ LT$$
$$V = 0.085\ (5{,}850)\ (1.42) = 706 \text{ acre-feet}$$

Example 2.--For the basin previously described, assume that an existing drainage structure will only pass a discharge of 4,050 cubic feet per second (25-year flood, approximate) before road overflow begins. Also, assume an estimate of the duration of road overflow resulting from the 100-year flood (Q_p = 5,850 cubic feet per second) is needed for risk-analysis considerations at the site. The duration of road overflow can be estimated from the hydrograph width (W) using figure 11 or table 6.

Solution:

(1) Q/Q_p = 4,050/5,850 = 0.69;

from figure 11, W/LT = 0.59; for Q/Q_p = 0.69

(2)From example 1, basin lagtime (LT) = 1.42 hours;
duration of road overflow = (W/LT) (LT)
= (0.59) (1.42)
= 0.84 hour or about 50 minutes

Example 3.--Simulate flood hydrographs that might be expected before and after projected urban development of an ungaged basin having a drainage area (A) of 7.5 square miles. Assume that hydrographs for floods having a 50-year recurrence interval are of interest for a rural condition, a condition of partial urban development, and a condition of intensive urban development. Percentages of impervious area (I) for these conditions are assumed to be 1, 10, and 25 percent.

Table 8.--*Computation of coordinates for the simulated hydrograph of the 100-year flood in application example 1*

[T, time, in hours; LT, basin lagtime, in hours; Q, discharge, in cubic feet per second; Q_p, peak discharge, in cubic feet per second]

T/LT (table 3)	x	LT	=	T	Q/Q_p (table 3)	x	Q_p	=	Q
0.25		1.42		0.36	0.11		5,850		644
.30		1.42		.43	.14		5,850		819
.35		1.42		.50	.18		5,850		1,050
.40		1.42		.57	.23		5,850		1,350
.45		1.42		.64	.29		5,850		1,700
.50		1.42		.71	.37		5,850		2,160
.55		1.42		.78	.46		5,850		2,690
.60		1.42		.85	.55		5,850		3,220
.65		1.42		.92	.65		5,850		3,800
.70		1.42		.99	.74		5,850		4,330
.75		1.42		1.07	.83		5,850		4,860
.80		1.42		1.14	.89		5,850		5,210
.85		1.42		1.21	.95		5,850		5,560
.90		1.42		1.28	.98		5,850		5,730
.95		1.42		1.35	1.00		5,850		5,850
1.00		1.42		1.42	.98		5,850		5,730
1.05		1.42		1.49	.95		5,850		5,560
1.10		1.42		1.56	.90		5,850		5,260
1.15		1.42		1.63	.84		5,850		4,910
1.20		1.42		1.70	.77		5,850		4,500
1.25		1.42		1.78	.71		5,850		4,150
1.30		1.42		1.85	.65		5,850		3,800
1.35		1.42		1.92	.59		5,850		3,450
1.40		1.42		1.99	.53		5,850		3,100
1.45		1.42		2.06	.48		5,850		2,810
1.50		1.42		2.13	.44		5,850		2,570
1.55		1.42		2.20	.40		5,850		2,340
1.60		1.42		2.27	.37		5,850		2,160
1.65		1.42		2.34	.34		5,850		1,990
1.70		1.42		2.41	.31		5,850		1,810
1.75		1.42		2.49	.28		5,850		1,640
1.80		1.42		2.56	.26		5,850		1,520
1.85		1.42		2.63	.24		5,850		1,400
1.90		1.42		2.70	.22		5,850		1,290
1.95		1.42		2.77	.20		5,850		1,170
2.00		1.42		2.84	.19		5,850		1,110
2.05		1.42		2.91	.17		5,850		995
2.10		1.42		2.98	.16		5,850		936
2.15		1.42		3.05	.15		5,850		878
2.20		1.42		3.12	.14		5,850		819
2.25		1.42		3.20	.13		5,850		761
2.30		1.42		3.27	.12		5,850		702
2.35		1.42		3.34	.11		5,850		644
2.40		1.42		3.41	.10		5,850		585

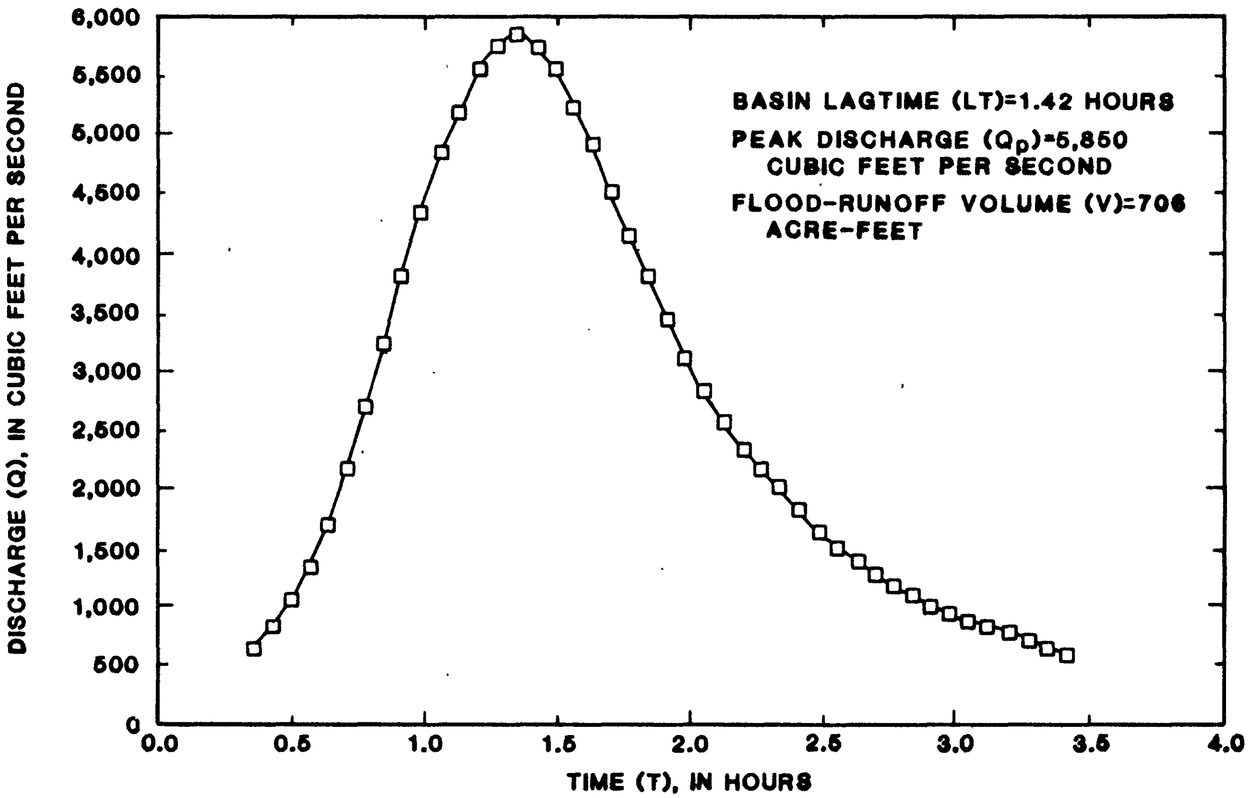


Figure 12.--Simulated hydrograph for the 100-year flood in application example 1.

Solution:

(1) Compute estimates of basin lagtime for A = 7.5 and I = 1, 10, and 25 using equation 1 (table 4).

$$LT = 1.46\, A^{0.34}\, I^{-0.19}$$

By substitution:

for A = 7.5 and I = 1, LT = 2.90 hours,
for A = 7.5 and I = 10, LT = 1.87 hours, and
for A = 7.5 and I = 25, LT = 1.57 hours.

(2) Compute estimates of peak discharge for 50-year floods using equation 12 from Supplemental Data section.

$$Q_{50} = 855\, A^{0.810}\, I^{0.137}$$

By substitution:

for A = 7.5 and I = 1, Q = 4,370 cubic feet per second,
for A = 7.5 and I = 10, Q = 5,990 cubic feet per second, and
for A = 7.5 and I = 25, Q = 6,800 cubic feet per second.

(3) Compute time (T) and discharge (Q) coordinates for simulated flood hydrographs for each of the three conditions of impervious area (I = 1, 10, and 25 percent) by expansion of the dimensionless hydrograph (table 3). Computations of time and discharge coordinates are not shown; however, resultant simulated hydrographs are shown in figure 13.

SUMMARY

This study was directed toward development of a technique for simulating flood hydrographs for small rural and urban basins in Missouri. The information is needed for planning and designing drainage structures, including risk analysis, and for other uses, such as establishing equitable land-use regulations.

Sufficient data were available from streamflow-gaging stations operated during previous studies to provide the flood flow information needed for reliable analyses. Data used in this study were those resulting from past flood-frequency investigations. Analyses of data from as many as 61 streamflow-gaging stations resulted in the development of a simple, practical technique for simulating flood hydrographs and of equations for estimating basin lagtimes and flood-runoff volumes at ungaged sites on small rural and urban drainage basins in Missouri.

Several flood-hydrograph-simulation methods were investigated; however, a dimensionless-hydrograph method developed by the U.S. Geological Survey was used. Hydrographs for 341 floods at 41 streamflow-gaging stations on small rural and urban basins in Missouri were analyzed. A dimensionless hydrograph was developed for Missouri that closely approximates the dimensionless hydrograph developed for Georgia, which has been verified in other areas.

These analyses have provided: (1) A dimensionless hydrograph that can be used for simulation of flood hydrographs at ungaged sites, (2) equations for estimating basin lagtimes, and (3) equations for estimating flood-runoff volumes. Coordinates for a simulated flood hydrograph can be computed by expansion of the dimensionless hydrograph developed for Missouri using basin lagtime and peak discharge for a flood with a specified recurrence interval.

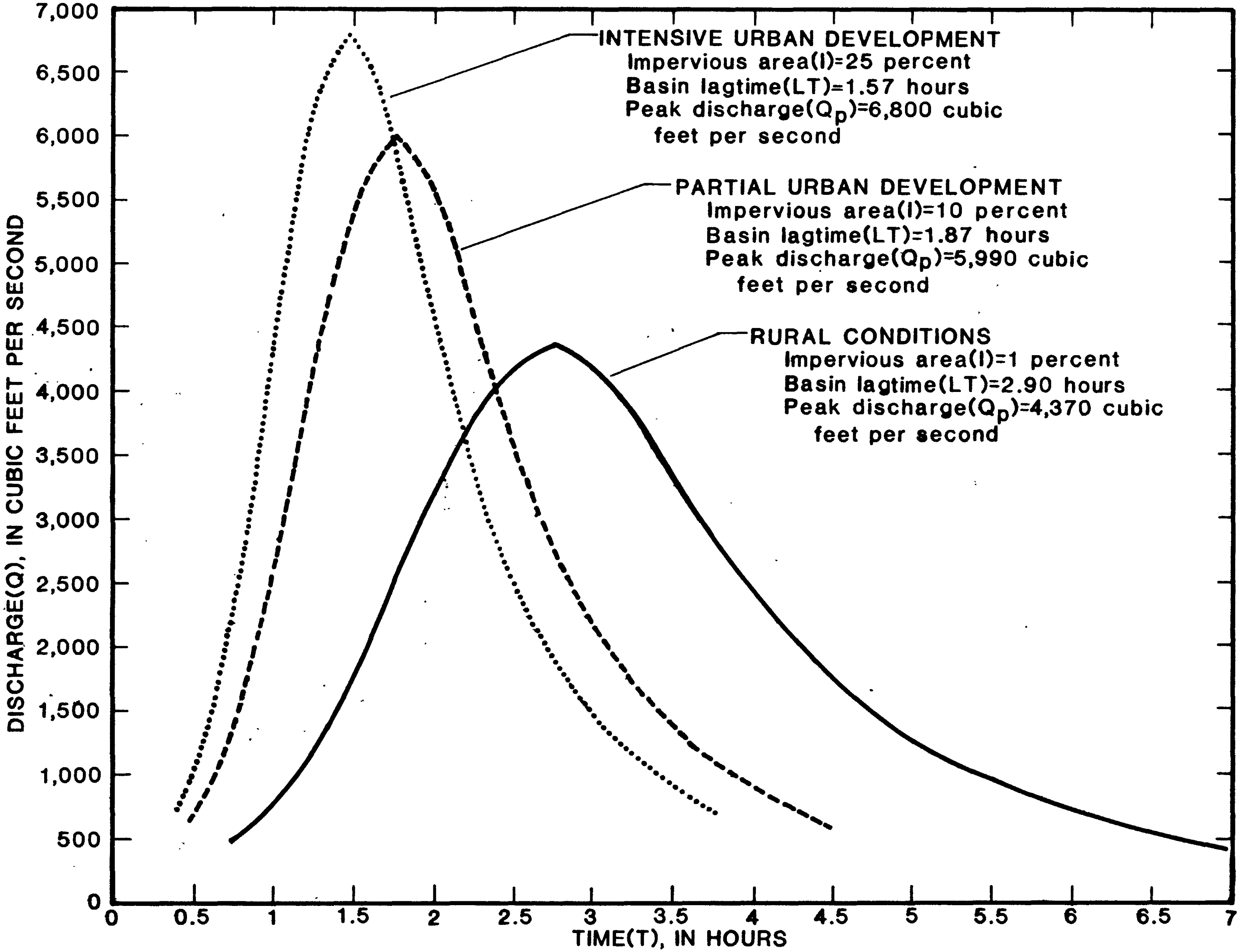


Figure 13.--Simulated flood hydrographs for three different conditions of urbanization (impervious area) of the drainage basin in application example 3.

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SUPPLEMENTAL DATA

Flood-Frequency Equations for Small Basins in Missouri

Estimates of peak discharges, in cubic feet per second, for floods having a 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval can be computed for small rural and urban basins in Missouri by using one of the two following sets of equations (Becker, 1986, p. 20). Alternative sets of equations, of approximately equal accuracy, are provided for convenience of the user. The equations for estimating peak discharge are considered applicable to contributing drainage areas ranging from about 0.25 to about 40 square miles. Estimates for rural basins may be made by assuming a minimum value of 1 percent for impervious area (I) in equations relating peak discharge (Q) to drainage area (A) and impervious area (I).

Equations for peak discharges, based on contributing drainage area (A) and percentage of impervious area (I), and the standard errors of estimate for these equations are:

Equation number	Peak-discharge equation	Standard error of estimate (percent)
(8)	$Q_2 = 224\ A^{0.793}\ I^{0.175}$	±32.3
(9)	$Q_5 = 424\ A^{0.784}\ I^{0.131}$	±29.5
(10)	$Q_{10} = 560\ A^{0.791}\ I^{0.124}$	±28.6
(11)	$Q_{25} = 729\ A^{0.800}\ I^{0.131}$	±27.2
(12)	$Q_{50} = 855\ A^{0.810}\ I^{0.137}$	±26.1
(13)	$Q_{100} = 986\ A^{0.821}\ I^{0.144}$	±25.9

Alternative equations for peak discharges, based on contributing drainage area (A) and basin development factor (BDF), and the standard errors of estimate for these equations are:

Equation number	Peak-discharge equation	Standard error of estimate (percent)
(14)	$Q_2 = 801\ A^{0.747}\ (13 - BDF)^{-0.400}$	±32.9
(15)	$Q_5 = 1{,}150\ A^{0.746}\ (13 - BDF)^{-0.318}$	±29.4
(16)	$Q_{10} = 1{,}440\ A^{0.755}\ (13 - BDF)^{-0.300}$	±28.4
(17)	$Q_{25} = 1{,}920\ A^{0.764}\ (13 - BDF)^{-0.307}$	±27.3
(18)	$Q_{50} = 2{,}350\ A^{0.773}\ (13 - BDF)^{-0.319}$	±26.5
(19)	$Q_{100} = 2{,}820\ A^{0.783}\ (13 - BDF)^{-0.330}$	±26.4

Determining the Basin Development Factor

The basin development factor (BDF) may be determined by using the methods described in the following excerpt from Sauer and others (1983) and the schematic shown in figure 14 (from Sauer and others, 1983, p. 7).

"The ***basin development factor (BDF***provides a measure of the efficiency of the drainage system. This parameter***can be easily determined from drainage maps and field inspections of the drainage basin. The basin is first divided into thirds***. Then, within each third, four aspects of the drainage system are evaluated and each assigned a code as follows:

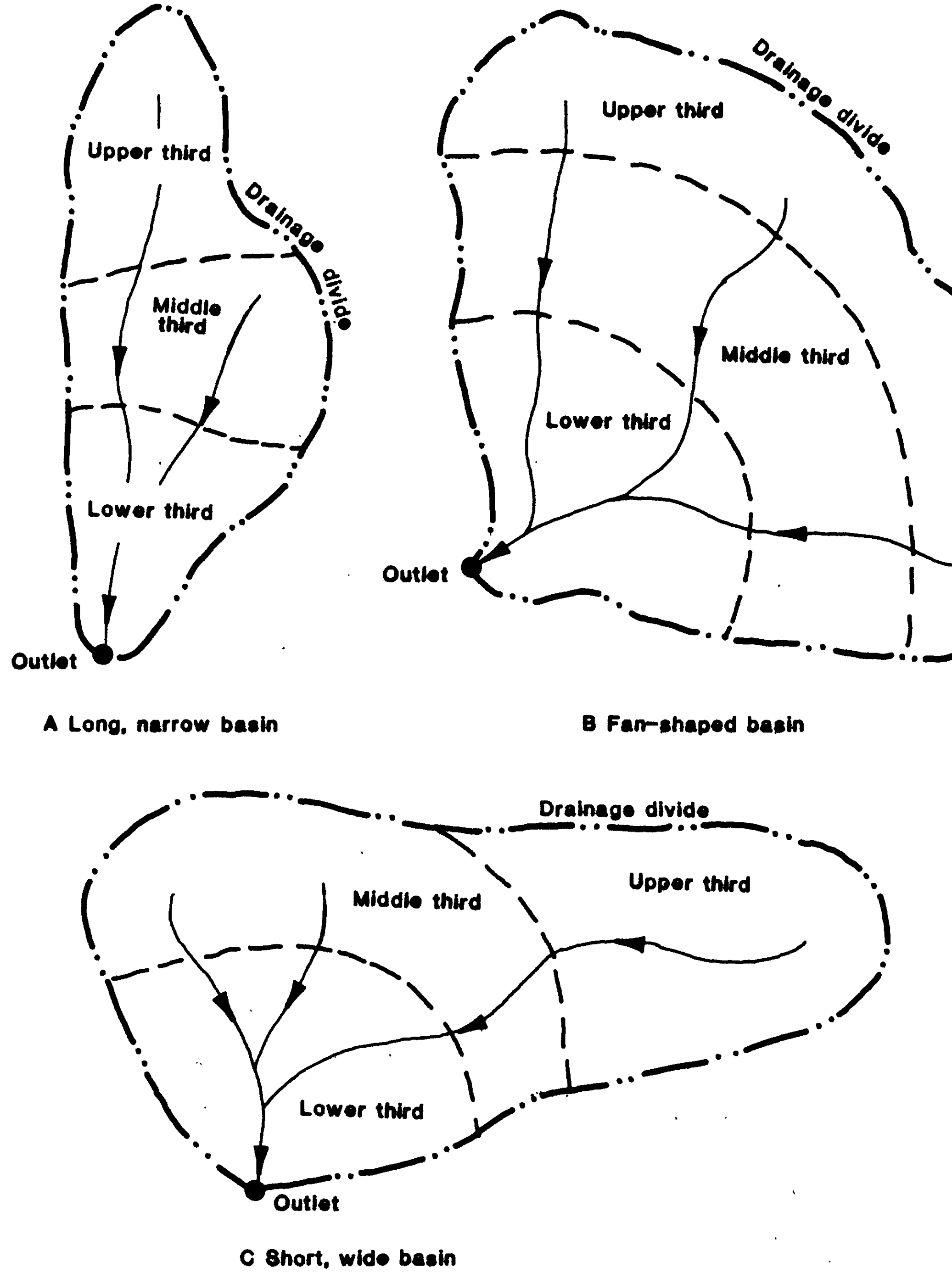

Figure 14.--Schematic of typical drainage-basin shapes and subdivision of the basins into thirds.

1. Channel improvements.--If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a code of 1 is assigned. Any or all of these improvements would qualify for a code of 1. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of zero is assigned.

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

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

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

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

The BDF is a fairly easy index to estimate for an existing urban basin. The 50-percent guideline will usually not be difficult to evaluate because many urban areas tend to use the same design criteria, and therefore have similar drainage aspects, throughout. Also, the BDF is convenient for projecting future development. Obviously, full development and maximum urban effects on peaks would occur when BDF = 12. Projections of full development or intermediate stages of development can usually be obtained from city engineers."