Effects of Urban Development on Floods in Northern Virginia

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 2001-C

Prepared in cooperation with the County of Fairfax and the city of Alexandria

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By DANIEL G. ANDERSON

WATER IN THE URBAN ENVIRONMENT

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A study of the influence of urbanization on peak discharges

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EXPLANATION OF SYMBOLS

A Drainage area, in square miles.
B Drainage class, basin designated as developed; all small tributaries storm sewered, but larger channels retained in a natural state.
d A subscript referring to a developed condition within a basin.
E Precipitation excess.
e A subscript referring to rainfall excess.
I The percentage of impervious area of a basin.
i A subscript referring to a given percentage of imperviousness within a basin.
K Coefficient that accounts for the percentage of impervious cover in the discharge equations.
L Length of the stream, in miles.
N Drainage class, basin designated as natural (rural).
n A subscript referring to a natural basin.
P Drainage class, basin designated as partially developed; some sewers and 4–20 percent imperviousness.
$\bar{Q}$ Average or mean annual flood, in cubic feet per second.
Q Peak discharge for basin condition or recurrence interval specified by subscript.
R Ratio of a flood to the mean annual flood in terms of the percentage of impervious area and frequency of the flood.
S The bed slope of the stream, in feet per mile.
T Lag time, in hours, defined as average time between centroid of rainfall excess and centroid of direct runoff.
U Drainage class, basin designated as fully developed; all streams completely sewered and main channels alined (repositioned in places to eliminate excessive meanders).
u A subscript referring to a completely sewered and fully developed basin.
z A subscript referring to a completely sewered basin without impervious area.

EFFECTS OF URBAN DEVELOPMENT ON FLOODS IN NORTHERN VIRGINIA

By Daniel G. Anderson

ABSTRACT

Graphical and mathematical relations are presented to estimate the flood-peak magnitudes having recurrence intervals ranging up to 100 years for drainage basins with various degrees of urban or suburban development. Five independent variables are required for use of the relations. They are the size, length, and slope of the basin, which may be measured from maps, and the percentage of impervious surface and type of drainage system, which may be evaluated by a basin inspection but in actual practice will usually be estimated for future developed conditions. Based upon analysis of flood information for 81 sites, 59 of which are in the Washington, D.C., metropolitan area, the relations should be useful for design of drainage systems and for definition of flood limits. The relations presented are applicable only to the Washington, D.C., area, but the method of analysis is general and may be used for any area where the major floods result from rainfall.

Urban and suburban development are shown to affect floodflows to a significant degree. Improvements of the drainage system may reduce the lag time to one-eighth that of the natural channels. This lag-time reduction, combined with an increased storm runoff resulting from impervious surfaces, increases the flood peaks by a factor that ranges from two to nearly eight. The flood-peak increase depends upon the drainage-basin characteristics and the flood recurrence interval.

INTRODUCTION

PURPOSE AND SCOPE

Suburban areas in all parts of the Nation are growing at a remarkable rate. Streets, housing developments, and shopping centers are replacing farms and woodlands. Continued growth is expected and will increase competition for available space. Careful guidance and planning of future development will be required to insure optimum land use.

Effects of flooding are a necessary consideration in planning land use and development. Encroachment in flood-prone areas must be controlled, and adequate storm sewers and drainage channels must be provided at minimum cost. Municipal planners and engineers, therefore, need information on the expected frequency and discharge of floods and on the possible depths and areal extent of inundation.

Given the magnitude of an expected flood, engineers can design the drainage system needed or determine the areas subject to inundation. Hydrologists have defined reasonably accurate methods for estimating the magnitude and frequency of floods expected from drainage basins in a rural condition. However, as a drainage basin is changed to a suburban or urban condition, the magnitude and frequency of flooding also change. These changes resulting from basin development have received only limited study because of the small amount of information available.

In an effort to obtain guidelines for optimum land-use planning, the U.S. Geological Survey, in cooperation with Fairfax County and the city of Alexandria, Va., established a cooperative project to study the effects of basin development on floods. The project allowed for collection of basic data, for analytical investigation, and for definition of flood-prone areas.

This report describes the procedures used and the results obtained in an analysis of the effects of urbanization on flooding. Results are presented as mathematical and graphical relations that may be used to estimate, at sites having various degrees of development in the Washington metropolitan area, the flood-peak discharge expected to be exceeded, on the average, once in any period of time ranging up to 100 years. This is the information needed to design drainage systems and to delineate flood-prone areas. Although the cooperative project includes delineation of flood-prone areas, that phase is considered to be a separate problem, unique for each stream, and is omitted from discussion in this report.

The relations presented are applicable only to the Washington, D.C., area, but the method of analysis is general and may be used for any area where the major floods result from rainfall.

ACKNOWLEDGMENTS

This report was prepared under the supervision of Mr. J. W. Gambrell, District Chief of the U.S. Geological Survey, Water Resources Division. Assistance in obtaining and analyzing data was received from many individuals and organizations. Dr. John Geyer, of the Johns Hopkins University, and Dr. Donald Dean, of the University of Delaware, allowed use of unpublished data collected by their institutions. Many hours of help in the reduction and interpretation of these data were provided by Dr. John Schaake, then of the Johns

Hopkins University. Mr. Donald S. Wallace, of the Virginia Department of Conservation and Economic Development, Water Resources Division, made available streamflow records collected by that organization. Officials of Fairfax County and the city of Alexandria furnished information valuable to the project.

DESCRIPTION OF AREA

Fairfax County and the city of Alexandria are adjacent to and west of Washington, D.C. (fig. 1). Alexandria has been a commercial and residential city since colonial times and in recent years has undergone considerable development and redevelopment. A large part of Fairfax County remains in a rural state, primarily pasture or woodland, although residential development is occupying increasingly larger parts. The population of Fairfax County has grown from 41,000 in 1940 to

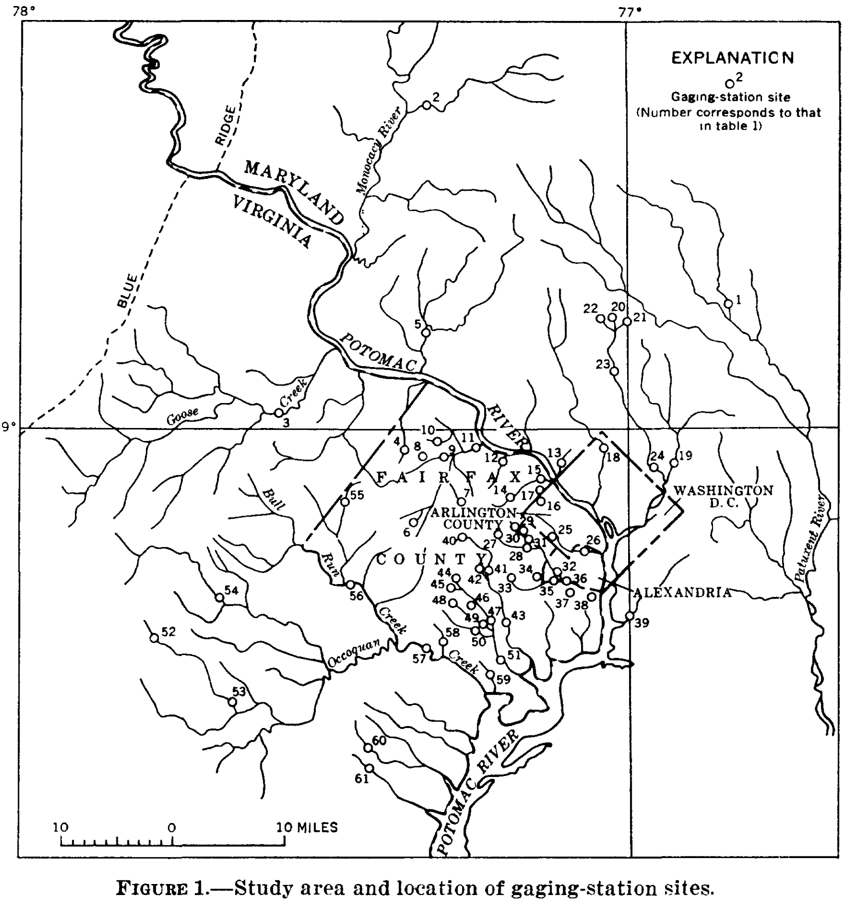

FIGURE 1.—Study area and location of gaging-station sites.

360,000 in 1966 and is expected to reach 1 million before the end of the century.

The study area is in the Piedmont and Coastal Plain physiographic provinces (Fenneman, 1938). Land surfaces are less than 500 feet above sea level and are gently rolling. Runoff reaches the Potomac River through small streams; only Occoquan Creek drains more than 100 square miles. Stream channels are well defined, and most flood plains are covered with a dense growth of brush. Stream channels are fairly steep; main channels commonly fall over 15 feet per mile, and small tributary streams commonly fall over 100 feet per mile.

The climate is humid. Average annual precipitation exceeds 40 inches and is about uniformly distributed throughout the year. Three types of storms can cause flooding in the area. Summer thunderstorms having short-duration but high-intensity precipitation are the most frequent cause of flooding. Longer duration but lower intensity precipitation resulting from frontal storms may occur in any season and occasionally cause flooding. Hurricane storms in late summer and fall may cause severe flooding.

PREVIOUS STUDY

Analytical methods employed in this report are based upon concepts presented by Carter (1961), described with the aid of figure 2, a symbolic representation (not to scale) of flood hydrographs. These concepts are applicable to average-size floods.

The solid-line hydrograph in figure 2 represents a flood from a natural drainage basin. If the drainage system of the basin is improved by adding storm sewers and alining stream channels,[1] the runoff will leave the basin faster and thereby change the hydrograph to a shape shown by the dash-dot line. Storm-sewer drainage is assumed to have negligible effect on the volume of precipitation excess, so the volume of runoff represented by the area under the hydrograph is equivalent for these two conditions. As a drainage basin is developed construction of buildings, roads, and parking lots reduces the amount of precipitation that infiltrates into the ground, thereby increasing the amount of precipitation excess or direct runoff; this results in the hydrograph shown by the broken line. Discharge-hydrograph changes shown in figure 2 are based on the assumption of uniform areal distribution of development within the basin. If development

[1] In this report, alining a stream channel means repositioning it in places to eliminate excessive meanders.

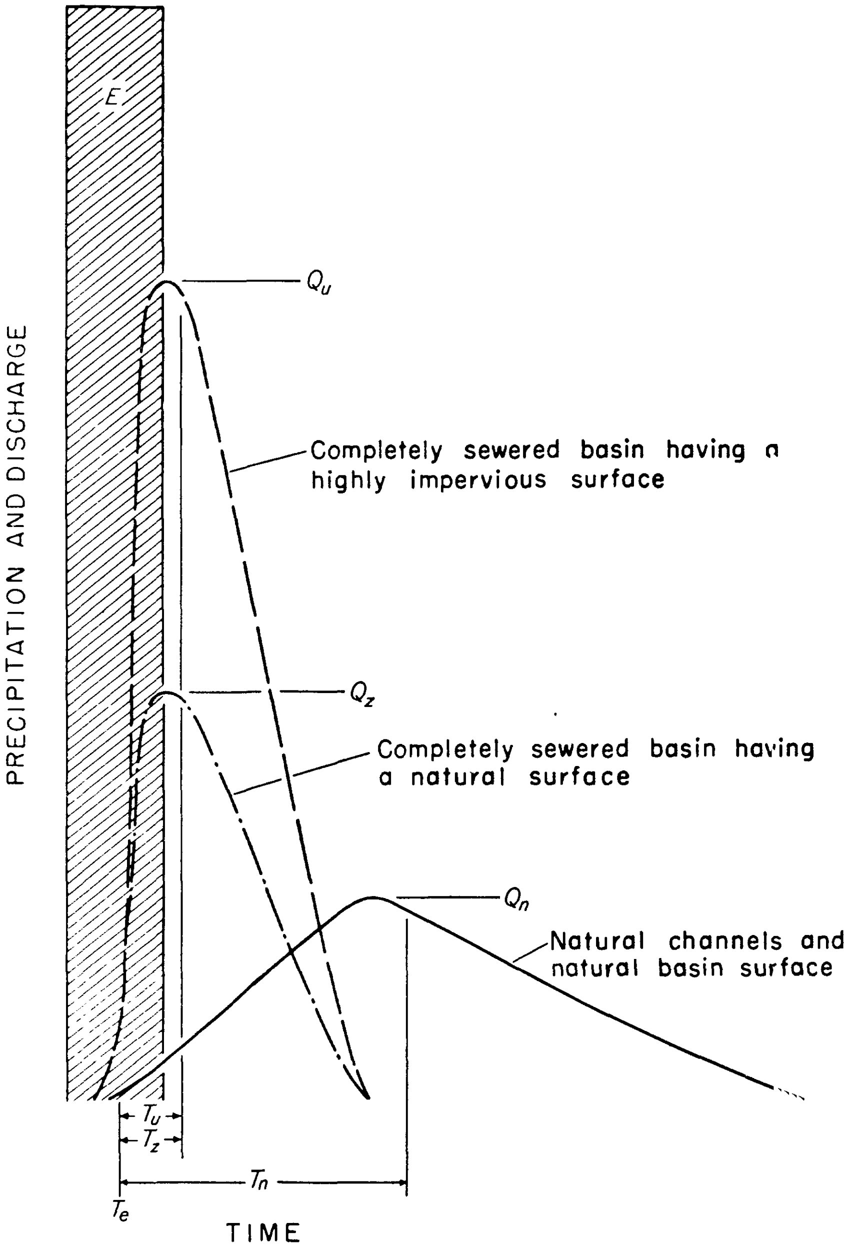

FIGURE 2.—Flood hydrographs. Hydrographs are not to scale. E, precipitation excess; T_e, time of the centroid of precipitation excess; T_n, T_z, and T_u, lag times (see definition on p. C6) ; Q_n, Q_z, and Q_u, flood peaks of the hydrographs for the three basin types shown.

occurred only in the lower or upper part of the basin, very different hydrographs might result. In actual basin developments, the drainage-channel improvements and impervious-area construction occur concurrently, and there is little opportunity to measure the effects of either change independently.

Carter (1961) proposed a coefficient of imperviousness, K, to evaluate the change in runoff and peak flows, so that $Q_u = KQ_s$, where Q_u is the peak discharge after ultimate development and Q_s is the peak discharge for a completely sewered basin without impervious areas. He suggested that the value of K could be determined from the percentage of basin area covered with impervious surface. From a study of rainfall quantities and flow volumes, he determined that for average floods in the Washington metropolitan area about 30 percent of total rainfall on natural basins becomes direct runoff; and he assumed that about 75 percent of rainfall on impervious surfaces becomes direct runoff. On this basis he formulated the coefficient of imperviousness, K, as

$$K = \frac{0.30 - 0.003I + 0.0075I}{0.30} = 1.00 + 0.015I, \qquad (1)$$

where I is the percentage of basin area covered with impervious surface

The change in peak discharge because of drainage improvements is related to lag time, T. Lag time is defined as the time from the centroid of rainfall excess to the centroid of direct runoff and is shown diagrammatically in figure 2. Synder (1958) found that lag time could be determined from indices of basin length, L; basin slope, S; and channel roughness. Carter used length and slope as indices to estimate separate lag times for basins with natural channels and with all channels sewered.

For natural conditions, the between-basins variation of average-flood size, $\overline{Q}$, is related to basin area, A, and to lag time, T. A mathematical expression of this relation may be determined by regression techniques. Carter noted that data from developed basins may be used to aid in determining the relation if the effects of imperviousness are removed and it is assumed that T and K are independent. On this basis he defined a relation of the form

$$\frac{\overline{Q}}{K} = f(A, T)$$

and suggested that by determining appropriate values for K, A, and T, the relation could be used to estimate average-size floods for basins with impervious surfaces and storm-drainage systems.

COLLECTION OF DATA

The effects of urban and suburban development on average floodflows can be defined from data collected in natural and developed basins using the analytical techniques described earlier. Precipitation and streamflow information gathered from the regular gaging-station network in northern Virginia was inadequate to define these effects with confidence. Therefore, a supplemental data-collection system was designed and started in 1959. A network could have been designed which would monitor basins as they changed from a natural to a developed condition; however, complete basin development can and often does require many years. The simplest and most expedient approach, therefore, was to sample for a short period many basins having a wide range of physical characteristics and basin development.

Streamflow information gathered from the supplemental network was a continuous record of flow during flood periods. Water-stage recorders were installed in simple structures, each consisting of a plywood shelter mounted on a corrugated pipe well. Discharge measurements were made to determine rating curves needed to convert the recorded water stage to a continuous record of floodflow. Because of the simple method of gage construction, a gaging station could easily be moved to a new site after adequate information had been obtained. The regular gaging-station programs maintained by the U.S. Geological Survey in Maryland and Virginia provided data for 17 sites, and the supplemental gaging network provided streamflow data for 38 sites. Data for six regular gaging stations were provided by the Virginia Department of Conservation and Economic Development, Water Resources Division.

Additional streamflow information for 14 small highly impervious basins were obtained from personnel of the Johns Hopkins University in Baltimore, Md., and the University of Delaware in Newark, Del. Lag times for six completely sewered streams in Louisville, Ky., were obtained from a report by Snyder (1958).

To supplement the precipitation records gathered by the U.S. Weather Bureau, tipping-bucket rain gages were installed at all regular and supplemental gaging sites. These gages recorded the time distribution of rainfall directly on the water-stage-recorder charts and provided an accurate measure of the lag time between precipitation excess and resulting runoff.

The data-collection network provided for this study the most complete information that has yet been obtained for a study of the effects of suburban development on floods.

ANALYSIS

The analysis consists of two parts. In the first part, methods proposed by Carter (1961) are used and the increased amount of information now available employed to verify or adjust relations he had defined for evaluating the effect of basin development on average-size floods. Relations are developed for estimating the average-size flood from basin characteristics for drainage areas in a rural condition and for urban or suburban development. In the second part of the analysis, relations are defined for evaluating the effects of urbanization on floods of all sizes.

DETERMINATION OF BASIN CHARACTERISTICS

Lag time, coefficient of imperviousness, and average flood were related to basin characteristics. All drainage basins of small to moderate size lying within about 50 miles of the Alexandria-Fairfax area were used to develop these relations. The sample basins ranged in character from almost entirely natural to completely paved areas with storm sewers replacing all natural channels.

Indices of the physical and hydrologic characteristics were determined for each basin and are shown in table 1. In table 1, columns 1 and 2 show, respectively, the numbers used to identify stream-gage locations in figure 1 and those used to identify streamflow records appearing in U.S. Geological Survey data publications. Columns 3 through 5 indicate the drainage basin, the period of streamflow records, and the agency that collected the data. Basin indices are shown in columns 6–13 and were evaluated by methods described in the following paragraphs.

Basin size, A, is the drainage basin area in square miles. For most sites it was determined by planimetering the basin outline on the best available topographic maps. For very small basins the drainage boundaries were surveyed, and the area was planimetered from a large scale drawing.

Lag time, T, is the average time interval between the centroid of rainfall excess and the centroid of direct runoff. Lag time is fairly constant for all average-size or larger floods from both natural and uniformly developed drainage areas, provided that the basin was wet prior to the storm.

Computation of lag time is a tedious job. To facilitate the work, lag time was determined from the three to six storms on each basin which came closest to meeting the following criteria:

1. Rainfall records indicated approximately uniform areal distribution of rainfall.
2. Rainfall duration was short compared to lag time.

TABLE 1.—*Characteristics of drainage basins used in analysis*

[Data source: USGS, U.S. Geol. Survey; VDWR, Virginia Div. Water Resources; JHU, the Johns Hopkins University; UD, Univ. Delaware; S, Snyder (1958). Drainage class: see symbols, p. IV]

Reference No. (fig. 1)	U.S. Geological Survey station	Stream and location	Period of record used	Data Source	Basin characteristics: Area (sq mi)	Lag time (hours)	Imperviousness (percent)	Length (miles)	Slope (ft/mi)	Length-slope $L/\sqrt{S}$	Drainage class	Average flood $\bar{Q}$ (cfs)	Coefficient of imperviousness K
1	1-5935.00	Little Patuxent River at Guilford, Md	1932-62	USGS	38.0	----	----	----	----	----	N	----	1.00
2	1-6425.00	Linganore Creek near Frederick, Md	1934-62	USGS	82.3	----	----	----	----	----	N	----	1.00
3	1-6440.00	Goose Creek near Leesburg, Va	1909-12 1930-66	USGS	338	14.7	<1	41.7	7.5	15.2	N	6,900	1.00
4	1-6443.00	Sugarland Run at Herndon, Va	1965-66	USGS	3.36	4.2	<1	2.6	38.9	.42	N	----	1.00
5	1-6450.00	Seneca Creek at Dawsonville, Md	1930-66	USGS	101	7.0	<1	21.6	15.1	5.6	N	2,400	1.00
6	1-6457.50	South Fork Little Difficult Run near Fairfax, Va.	1967	USGS	1.59	3.0	<1	3.0	51.2	.43	N	----	1.00
7	1-6458.00	Piney Branch at Vienna, Va	1963-66	USGS	.29	.18	30	.5	86.5	.053	B	210	1.45
8	1-6458.70	Colvin Run tributary at Reston, Va	1963	USGS	1.06	1.9	<1	1.6	84.9	.17	N	----	1.00
9	1-6459.00	Colvin Run at Reston, Va	1961-66	USGS	5.09	3.4	<1	3.7	49.3	.52	N	----	1.00
10	1-6459.50	Piney Run at Reston, Va	1965-66	USGS	2.06	2.6	----	1.3	79.6	.14	N	----	1.00
11	1-6460.00	Difficult Run near Great Falls, Va	1935-66	VDWR	58	9.2	----	13.2	16.0	3.3	N	1,480	1.00
12	1-6462.00	Scott Run near McLean, Va	1961-66	USGS	4.69	1.6	5	4.2	54.0	.57	P	600	1.08
13	1-6465.50	Little Falls Branch near Bethesda, Md	1944-66	USGS	4.1	1.0	15	3.1	58.0	.41	P	1,100	1.22
14	1-6466.00	Pimmit Run near Falls Church, Va	1961-66	USGS	2.87	1.0	12	3.0	59.4	.39	P	430	1.18
15	1-6467.00	Pimmit Run at Arlington, Va	1961-66	USGS	8.12	3.0	12	7.2	38.7	3.0	P	1,000	1.18
16	1-6467.50	Little Pimmit Run tributary at Arlington, Va.	1962-66	USGS	.41	.32	28	.9	98.5	.09	B	340	1.42
17	1-6468.00	Little Pimmit Run at Arlington, Va	1961-66	USGS	2.31	.37	20	2.2	77.4	.26	B	1,000	1.30
18	1-6480.00	Rock Creek at Sherrill Drive at Washington, D.C.	1929-66	USGS	62.2	10.0	5	23.0	10.5	6.8	P	1,620	1.08
19	1-6495.00	Northeast Branch Anacostia River at Riverdale, Md.	1938-66	USGS	72.8	12.0	2	15.5	27.2	3.0	N	2,500	1.03
20	1-6500.50	Northwest Branch Anacostia River at Norwood, Md.	1966-67	USGS	2.45	2.3	3	2.0	46.7	.29	N	----	1.04
21	1-6500.85	Nursery Run at Cloverly, Md	1966-67	USGS	.35	2.0	<1	1.0	117	.093	N	----	1.00
22	1-6501.90	Batchellors Run at Oakdale, Md	1966-67	USGS	.47	2.2	<1	1.2	108	.12	N	----	1.00
23	1-6505.00	Northwest Branch Anacostia River near Colesville, Md.	1924-66	USGS	21.3	4.8	3	7.4	19.1	1.4	N	1,250	1.04
24	1-6510.00	Northwest Branch Anacostia River near Hyattsville, Md.	1938-60	USGS	49.4	4.6	7	18.5	20.9	4.0	P	2,300	1.10
25	1-6524.00	Long Branch at Arlington, Va	1961-66	USGS	.94	.50	30	2.1	81.2	.24	B	710	1.45
26	1-6525.00	Fourmile Run at Alexandria, Va	1961-66	USGS	14.4	1.3	20	7.8	42.5	1.2	P	3,000	1.30
27	1-6526.00	Holmes Run at Merrifield, Va	1960-66	USGS	2.70	3.3	10	2.8	69.5	.34	P	450	1.15

TABLE 1.—*Characteristics of drainage basins used in analysis*—Continued

Reference No. (fig. 1)	U.S. Geological Survey station	Stream and location	Period of record used	Data Source	Basin characteristics: Area (sq mi)	Lag time (hours)	Imperviousness (percent)	Length (miles)	Slope (ft/mi)	Length-slope $L/\sqrt{S}$	Drainage class	Average flood $\bar{Q}$ (cfs)	Coefficient of imperviousness K
28	1-6526.10	Holmes Run near Annandale, Va	1960-66	USGS	7.10	3.5	12	5.8	36.8	.96	P	630	1.18
29	1-6526.20	Tripps Run at Falls Church, Va	1960-66	USGS	1.78	.43	25	2.3	79.2	.26	B	720	1.38
30	1-6526.45	Tripps Run tributary near Falls Church, Va.	1963-66	USGS	.50	.32	25	1.1	102	.11	B	280	1.38
31	1-6526.50	Tripps Run near Falls Church, Va	1960-66	USGS	4.55	.78	25	4.1	52.0	.56	B	1,230	1.38
32	1-6526.90	Holmes Run at Alexandria, Va	1960-61	USGS	18.9	5.8	12	10.7	31.3	1.9	P	----------	1.18
33	1-6527.10	Backlick Run at Springfield, Va	1960-66	USGS	2.02	1.1	15	2.3	50.3	.33	P	660	1.22
34	1-6528.10	Turkeycock Run at Alexandria, Va	1960-64	USGS	2.26	1.3	8	2.8	78.2	.31	P	430	1.12
35	1-6529.10	Backlick Run at Alexandria, Va	1960-66	USGS	13.4	2.0	10	7.1	28.9	1.3	P	1,970	1.15
36	1-6530.00	Cameron Run at Alexandria, Va	1955-66	USGS	33.7	4.1	15	11.1	30.9	2.0	P	3,400	1.22
37	1-6530.07	Pike Branch at Alexandria, Va	1960-64	USGS	2.65	.78	12	2.5	75.4	.29	P	750	1.18
38	1-6532.00	Penn Daw Outfall at Alexandria, Va	1963-66	USGS	.82	.48	20	1.5	158	.12	P	230	1.30
39	1-6535.00	Henson Creek near Oxon Hill, Md	1948-60	USGS	16.7	5.4	3	8.4	23.4	1.7	N	1,080	1.04
40	1-6539.00	Accotink Creek at Fairfax, Va	1961-66	USGS	6.80	2.0	10	4.7	35.9	.79	P	1,070	1.15
41	1-6540.00	Accotink Creek near Annandale, Va	1947-66	VDWR	23.6	6.8	8	11.2	18.9	2.6	P	1,500	1.12
42	1-6545.00	Long Branch near Annandale, Va	1950-57 1959-66	USGS	3.71	2.9	<1	4.3	44.6	.64	N	410	1.00
43	1-6550.00	Accotink Creek near Accotink Station, Va.	1949-57 1960-61	USGS	37.0	9.4	5	17.1	14.9	4.4	P	1,650	1.08
44	1-6553.10	Rabbit Branch near Burke, Va	1961-62 1964-66	USGS	3.81	3.8	<1	3.4	44.2	.51	N	380	1.00
45	1-6553.30	Sideburn Branch near Fairfax Station, Va	1960-62	USGS	2.79	2.4	<1	2.7	49.3	.39	N	----------	1.00
46	1-6553.40	Pohick Creek tributary near Burke, Va	1964-66	USGS	.34	1.4	3	.68	149	.056	N	----------	1.04
47	1-6553.50	Pohick Creek near Springfield, Va	1961-66	USGS	15.0	5.4	<1	9.0	23.8	1.8	N	770	1.00
48	1-6553.60	Sangster Branch near Burke, Va	1963-64	USGS	.15	1.6	<1	.60	151	.049	N	----------	1.00
49	1-6553.70	Middle Run near Lorton, Va	1961-66	USGS	3.56	3.9	<1	4.0	44.5	.60	N	470	1.00
50	1-6553.80	South Run near Lorton, Va	1961-66	USGS	6.54	4.0	<1	6.7	29.3	1.2	N	500	1.00
51	1-6553.90	Pohick Creek at Lorton, Va	1961-66	USGS	31.0	6.9	<1	14.0	24.0	2.9	N	1,230	1.00

52	1-6555.00	Cedar Run near Warrenton, Va	1950-66	VDWR	13.0	2.3	<1	4.4	67.5	.72	N	1,100	1.00
53	1-6560.00	Cedar Run near Catlett, Va	1950-66	VDWR	93.5	10.6	<1	19.4	21.9	4.1	N	3,400	1.00
54	1-6565.00	Broad Run at Buckland, Va	1950-66	VDWR	50.3	6.8	<1	17.3	20.9	3.8	N	1,700	1.00
55	1-6568.00	Cub Run near Chantilly, Va	1963-66	USGS	7.13	4.8	<1	3.4	20.1	.77	N	350	1.00
56	1-6570.00	Bull Run near Manassas, Va	1950-66	VDWR	147	13.2	<1	20.8	10.0	6.6	N	7,000	1.00
57	1-6575.00	Occoquan Creek near Occoquan, Va	1913-16 1920-23 1937-56	USGS	570	18.6	<1	52.0	6.5	20.3	N	14,200	1.00
58	1-6576.00	Sandy Run near Farifax Station, Va	1966	USGS	2.35	3.3	<1	2.9	62.2	.36	N	----------	1.00
59	1-6578.00	Giles Run near Woodbridge, Va	1965-66	USGS	4.54	2.8	3	5.5	50.1	.78	N	----------	1.04
60	1-6585.00	South Fork Quantico Creek near Independent Hill, Va.	1951-66	USGS	7.50	6.0	<1	4.9	24.7	.98	N	550	1.00
61	1-6595.00	Middle Fork Chopawamsic Creek near Garrisonville, Va.	1951-57 1960-66	USGS	4.51	4.8	<1	4.7	43.2	.71	N	----------	1.00
62	----------	Grayhaven, Baltimore ,Md	1960-62	JHU	.036	.094	52	.25	67.7	.030	U	----------	1.79
63	----------	Hamilton Hills 1, Baltimore M,d	1963-66	JHU	.0011	.065	56	.09	43.8	.014	U	----------	1.84
64	----------	Hamilton Hills 3, Baltimore, Md	1963-66	JHU	.0029	.056	36	.11	44.8	.016	U	----------	1.54
65	----------	Hamilton Hills 4, Baltimore, Md	1962-64	JHU	.00034	.051	96	.11	45.4	.06	U	----------	2.44
66	----------	Hamilton Hills 5, Baltimore, Md	1962-64	JHU	.0027	.050	32	.08	111.	.0076	U	----------	1.48
67	----------	Montebello 3, Baltimore, Md	1961	JHU	.0007	.039	57	.03	42.7	.0046	U	----------	1.85
68	----------	Montebello 4, Baltimore, Md	1961	JHU	.0008	.044	65	.07	41.7	.010	U	----------	1.98
69	----------	Newark 9-inch flume, Newark, Del	1960-62	UD	.0006	.040	100	.11	177.	.0082	U	----------	2.50
70	----------	Newark 12-inch flume, Newark, Del	1960-62	UD	.0048	.065	100	.17	35.9	.029	U	----------	2.50
71	----------	Northwood, Baltimore, Md	1959-62	JHU	.074	.11	68	.39	213.	.027	U	----------	2.02
72	----------	Swansea, Baltimore, Md	1958-62	JHU	.074	.080	44	.41	215.	.028	U	----------	1.66
73	----------	Uplands, Baltimore, Md	1951-62	JHU	.047	.12	52	.42	166.	.032	U	----------	1.79
74	----------	Walker Ave., Baltimore, Md	1951-62	JHU	.24	.19	33	1.04	83.3	.11	U	----------	1.50
75	----------	Yorkwood South, Baltimore, Md	1958-62	JHU	.017	.080	41	.20	150.	.016	U	----------	1.61
76	----------	Beargrass Creek, Louisville, Ky	----------	S	6.30	.90	----------	4.0	4.5	1.9	U	----------	----------
77	----------	N. W. Trunk at Shawnee Park, Louisville, Ky.	----------	S	1.90	.60	----------	2.9	5.3	1.3	U	----------	----------
78	----------	17th Street at Northwestern Parkway, Louisville, Ky.	----------	S	.22	.38	----------	.9	16.9	.22	U	----------	----------
79	----------	Southern Outfall near State Fair Grounds, Louisville, Ky.	----------	S	6.43	1.1	----------	6.4	3.1	3.6	U	----------	----------
80	----------	Southwestern Outfall, Cane Road, near Camp Ground Road, Louisville, Ky.	----------	S	7.52	.83	----------	6.5	4.5	3.1	U	----------	----------
81	----------	Western Outfall, Broadway near Western Parkway, Louisville, Ky.	----------	S	2.77	.83	----------	4.2	2.6	2.6	U	----------	----------

3. Basin soils were wet before the storm.
4. The rainfall depth exceeded about half an inch.

For each selected storm, the volume of precipitation excess was adjusted to equal the volume of direct runoff. Methods described by Linsley, Kohler, and Paulhus (1958) were used to determine the time distribution of direct runoff from the hydrograph of total measured runoff. Infiltration curves similar to those described by Sherman (1940) aided in defining the time distribution of precipitation excess. The time difference between the centroids of these two distributions provided a measure of basin lag time.

Impervious-area index, I, is the percentage of basin area (A) covered with manmade impervious surfaces. Determinations for basins in the Washington metropolitan area were made using aerial photographs dating from 1956 and 1960. In this study, the impervious-area index represents conditions at the times for which the average-flood size and basin lag time were determined. The percentages of imperviousness for the basins in Maryland and Delaware were furnished by the agencies collecting the data.

Length, L, is the distance, in miles, along the primary water course from the basin mouth (stream-gaging site) to the basin boundary. It was measured on the best available maps.

Slope, S, is an index of basin slope. It was determined as the average slope, in feet per mile, of the main watercourse between points located 10 and 85 percent of the length, L, upstream from the stream gage. (Benson, 1959.)

Length-slope, $L/\sqrt{S}$, was computed as the ratio of basin length to the square root of the basin slope.

Drainage class is an arbitrary designation based on field inspection of drainage channels, percentage imperviousness of the basin, and the extent to which tributaries and main channels are served by storm sewers.

Average flood, $\overline{Q}$, was determined from a frequency curve. Frequency curves for each basin were constructed by computing the recurrence interval of each flood by the formula $(y+1)/m$ where m is the relative magnitude of the flood, the highest value being 1, and y is the number of years of record. A smooth curve through the plot of recurrence interval versus flood-peak discharge forms a frequency curve. For sites having 8 or more years of flood record, only the largest peak in each year (annual-flood series) was used, and $\overline{Q}$ was selected at a recurrence interval of 2.33 years. For sites with less than 8 years of record, all the largest floods (partial-duration series) were used, and $\overline{Q}$ was determined at the 1.78-year recurrence interval. Langbein (1949) found that $\overline{Q}$ for the partial-duration and annual-flood series were comparable at the 1.78- and 2.33-year recurrence intervals respectively. While this may not always be true (U.S. Army Corps of Engineers, 1960), analysis of long flood records in the study area indicates that Langbein's comparison is applicable.

DEFINITION OF RELATIONS

COEFFICIENT OF IMPERVIOUSNESS

The effect of building impervious surfaces on a drainage basin is to increase the volume of direct runoff and the magnitude of peak discharge. Values of K, the coefficient that accounts for this effect, were computed using the methods previously described. Information obtained during the study confirmed the validity of the method. A value of K was computed for each basin and is shown in table 1.

LAG TIME

Lag time changes as a basin experiences development; therefore an estimate of the lag time for the degree of expected basin development is required to estimate future flood conditions. Using data for 33 natural and 20 fully developed basins, relations were developed defining lag time, T, as a function of length and slope. Several linear-regression models were tried and the results are shown in table 2. The effectiveness of each relation was determined on the basis of its standard error of estimate, a measure of its accuracy. Approximately two-thirds of the estimates provided by an equation will be accurate within one standard error, and approximately 19 out of 20 estimates will be accurate within two standard errors. Although equations using $\log T = f(\log L, \log S)$ show a slightly smaller standard error, relations of the form $\log T = f(\log (L/\sqrt{S}))$ were selected as more appropriate for use on the basis of independent work by Snyder (1958) and theoretical considerations. Lag-time relations for natural channels (class N) and completely sewered channels (class U) are shown graphically in figure 3.

The ultimate degree of improvement predicted for most drainage systems in the Alexandria-Fairfax area is storm sewering of all small tributaries but retention of larger channels in a natural state. The

TABLE 2.—*Summary of relations for estimating lag time*

Functional expressions for log T	Natural basins		Fully developed basins	
	Expressions for T	Standard error of estimate, in percent	Expressions for T	Standard error of estimate, in percent
$f(\log L, \log S)$	$12.9\ L^{0.27}S^{-0.43}$	−20.5, +25.5	$0.47\ L^{0.56}S^{-0.20}$	−17.0, +20.6
$f(\log L)$	$1.78\ L^{0.55}$	−22.8, +29.4	$.26\ L^{0.60}$	−24.8, +31.9
$f(\log (L/\sqrt{S}))$	$4.64\ (L/\sqrt{S})^{0.42}$	−20.9, +26.0	$.56\ (L/\sqrt{S})^{0.52}$	−17.7, +21.3
$f(\log (L/S))$	8.34 $(L/S)^{0.3}$[illegible]	−20.2, +25.1	$.82\ (L/S)^{0.30}$	−24.8, +33.3

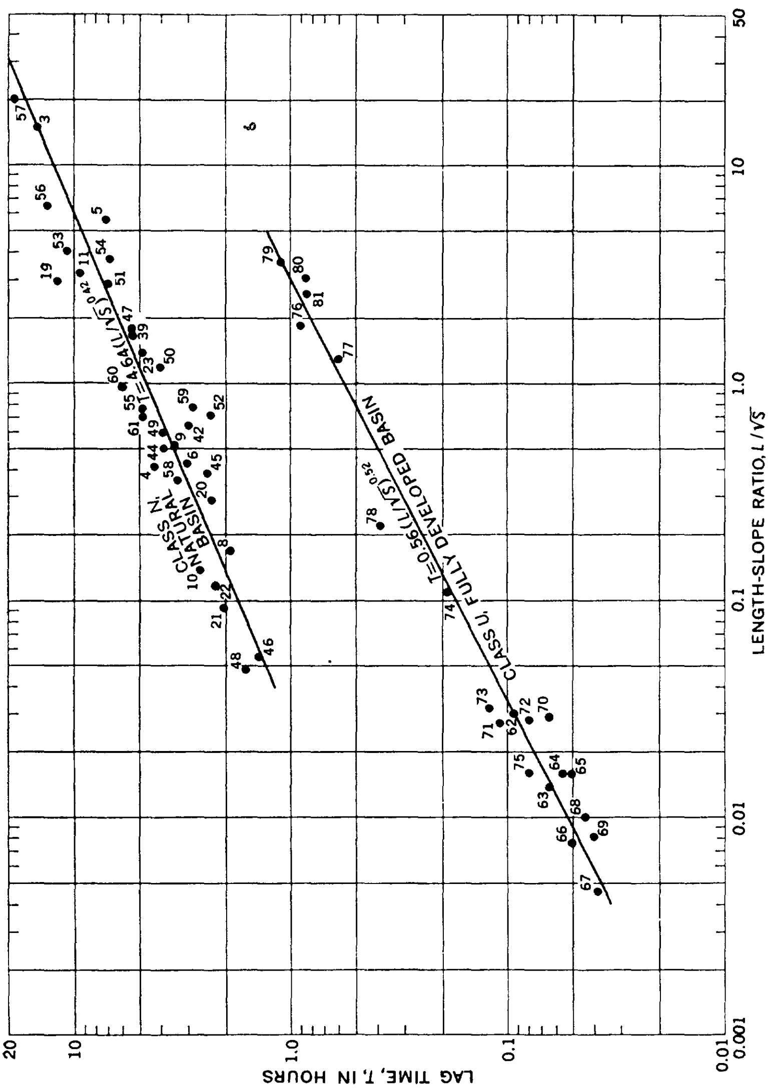

FIGURE 3.—Relation between lag time and basin length-slope ratio for natural (class *N*) and fully developed (class *U*) drainage basins. *L*, in miles. *S*, in feet per mile. Numbers for gaging-station sites correspond to those in figure 1 and table 1.

center relation shown in figure 4 provides estimates of lag time for this type (class *B*) of drainage system. The position of the center relation was based on plotted data for seven basins that are considered to have reached a condition of complete suburban development. The slope of the relation was computed by logarithmic interpolation between the slopes of the relations for natural (class *N*) and fully developed (class *U*) basins, which are also shown in figure 4.

AVERAGE-FLOOD MAGNITUDE

The basin-to-basin variation in size of the average flood, $\bar{Q}$, is related to variations in basin size, A, lag time, T, and coefficient of imperviousness, K. Using data for 44 of the basins shown in table 1 in a multiple-regression analysis, the relation was evaluated as

$$\frac{\bar{Q}}{K}=230A^{0.82}T^{-0.48}. \quad (2)$$

This relation has a standard error of −23.3 and +29.9 percent and is nearly identical with the one defined from limited data by Carter (1961).

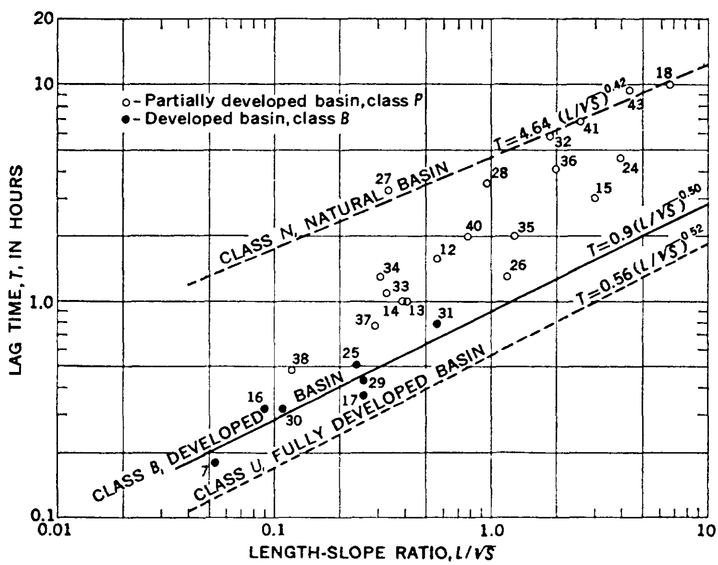

FIGURE 4.—Relations for estimating lag time for three classes of drainage systems. *L*, in mile; *S*, in feet per mile. Numbers for gaging-station site correspond to those in figure 1 and table 1.

By transposition of the variable K, the defined relation may be rewritten in the form

$$\bar{Q}=230KA^{0.82}T^{-0.48}. \tag{3}$$

In this form, the relation may be used to estimate the size of the average flood for any degree of basin imperviousness and for the three drainage systems shown in figure 4.

FREQUENCY AND MAGNITUDE

Previously defined relations provide a means of evaluating the average-size flood. In this section relations are described for estimating the magnitude of different flood frequencies and for various degrees of basin development. Frequency is expressed as a recurrence interval—the average length of time between occurrences of floods equal to or greater than the indicated size. For example, the flood size exceeded about five times each 100 years has a recurrence interval of 20 years.

A flood-frequency curve based on recorded flood events describes the flood experience of a basin. Dalrymple (1960) outlined methods of defining frequency curves as dimensionless ratios so that frequency curves of several basins could be compared. On the assumption that the difference in dimensionless frequency curves for natural basins in a local region resulted from vagaries of weather, Dalrymple proposed that the average dimensionless frequency curve for a region provides the best estimate of future flood frequencies. Methods outlined by Dalrymple were used to define the average dimensionless frequency relation for natural drainage basins in the Alexandria-Fairfax region.

A long flood record is required to define a flood-frequency curve accurately. In the study region, flood records for seven natural basins cover most of the period 1931–62 and were used to define the regional average relation. A frequency curve was drawn for each site using the largest flood in each year (annual-flood series). The magnitude scale of each curve was then converted to a scale of ratios by dividing by the average-flood magnitude (magnitude at 2.33-year recurrence interval). At selected recurrence intervals of 2.33, 10, 25, 50, and 100 years the ratios were determined and averaged as shown in table 3. These average ratios define the regional average-frequency relation for natural drainage basins shown as the upper curve in figure 5.

On an impervious surface the portion of rainfall becoming runoff is assumed to be constant for all storms. On a natural drainage basin a larger portion of rainfall from a large storm becomes flood runoff

TABLE 3.—*Flood-frequency ratios for basins with long flood records*

Stream and location	Recurrence interval, in years				
	2.33	10	25	50	100
Linganore Creek near Frederick, Md	1.00	1.48	1.74	1.92	2.10
Seneca Creek near Dawsonville, Md	1.00	2.17	3.35	4.65	6.52
Difficult Run near Great Falls, Va	1.00	1.78	2.44	3.06	3.75
Rock Creek at Sherrill Dr., Washington, D.C	1.00	2.36	3.64	4.78	6.07
Northwest Branch Anacostia River near Colesville, Md	1.00	2.18	3.25	4.05	5.00
Little Patuxent River at Guilford, Md	1.00	2.26	3.38	4.41	5.50
Piney Run near Sykesville, Md	1.00	2.09	3.24	4.57	6.52
Median ratios	1.00	2.17	3.25	4.41	5.50
Ratios used	1.00	2.2	3.3	4.4	5.5

than from a small storm. This variation of direct runoff with storm size indicates that the dimensionless frequency relations will differ for natural and for impervious basins. Insufficient streamflow data were available to define directly a regional flood-frequency relation for basins with varying degrees of imperviousness. It was assumed that the shape of a dimensionless frequency curve for impervious basins approaches the shape of a dimensionless rainfall-frequency relation as imperviousness approaches 100 percent. Using U.S. Weather Bureau rainfall-frequency relations (1955) the dimensionless ratios shown in table 4 were computed and considered applicable as flood-frequency ratios for 100-percent-impervious basins. These ratios define the lower curve in figure 5.

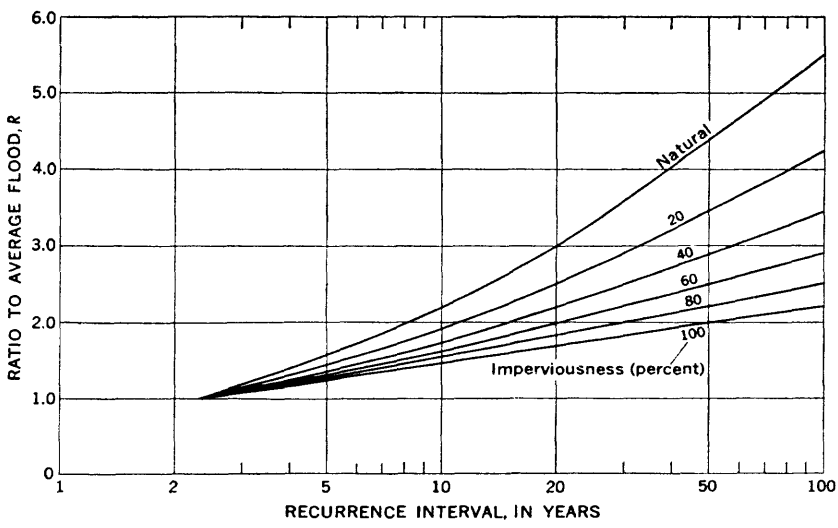

FIGURE 5.—Flood-frequency curves for selected degrees of basin imperviousness.

TABLE 4.—*Flood-frequency ratios used for natural and 100-percent-impervious drainage basins*

Recurrence intervals, in years	Flood-frequency ratios	
	Natural basins	100-percent-impervious basins
2.33	1.0	1.0
10	2.2	1.45
25	3.3	1.8
50	4.4	2.0
100	5.5	2.2

Flood-ratio curves in figure 5 for intermediate degrees of imperviousness were interpolated between the natural and 100 percent-impervious curves at any selected recurrence intervals using the relation

$$R_i = \frac{R_n + .01I(2.5R_{100} - R_n)}{1.00 + 0.015I}$$

where R_i is the flood ratio for a given percentage of imperviousness, I is the percentage of basin covered with impervious surface, R_{100} is the flood ratio for a 100-percent-impervious basin, and R_n is the flood ratio for a natural drainage basin. The curves of figure 5 provide a means of estimating the magnitude of a flood of any frequency if the average-flood size is known.

DISCUSSION

APPLICABILITY OF RELATIONS

A logical approach requiring several assumptions was used in defining the flood-estimation relations. The relations were based on more information than had hitherto been assembled for any urban area. Because regression analysis defines relations that are most accurate for the average of the sample data, flood estimates should be most accurate for basins with characteristics near the average of the sample basins. In particular the estimates are considered more accurate for basins in either a natural or a completely developed condition than for basins that are partially developed. Although the relations might be refined on the basis of information subsequently obtained, they should be useful to planners and engineers in their present form.

The relations were defined on the basis of information obtained primarily in the Washington, D.C., metropolitan area, and their applicability should be closely checked before they are used for any other area. Final computation of peak discharge is based on four factors: lag time, *T;* impervious coefficient, *K;* flood ratio, *R;* and drainage area, *A*.

Computation of the coefficient of imperviousness was based on runoff studies for natural and developed areas in northern Virginia. These relations may be far different in semiarid and arid areas where (1) the storm rain usually falls on dry soil and may run off in previously dry channels and (2) a high rate of evaporation may affect even the runoff from impervious surfaces.

Flood ratio *R* was based on interpolation between ratios for natural-flow runoff and those for 100-percent-impervious conditions derived from rainfall frequencies. The natural-flow ratios are for stations in the Washington metropolitan area. In other regions these ratios may vary drastically, primarily with climatic and rainfall characteristics.

EFFECTS OF DEVELOPMENT

The effects of development on flood-peak discharges may be assessed by using the defined relations in a ratio form

$$\frac{Q_d}{Q_n}=\frac{230K_dA^{0.82}T_d^{-0.48}R_d}{230K_nA^{0.82}T_n^{-0.48}R_n}=\frac{K_dR_d}{K_nR_n}\left(\frac{T_n}{T_d}\right)^{0.48},$$

where the subscripts *d* and *n* refer to developed and natural conditions respectively. Using this equation, the effects of three length-slope ratios were computed for selected degrees of basin development and several flood-recurrence intervals. Results are shown in table 5. The three types of basins considered are small and steep, average, and large and flat as represented by $L/\sqrt{S}$ values of 0.1, 1, and 10 respectively. The two types of developed drainage systems considered are (1) completely storm sewered basins having alined channels and (2) basins having sewered tributaries and unalined, natural main channels. Ratios were computed for 0-, 20-, 50-, and 100-percent imperviousness for floods having recurrence intervals of 2.33 (average), 25, 50, and 100 years.

TABLE 5.—*Flood-peak-magnitude ratios for developed basins (classes* U *and* B) *to natural basins (class* N)

Recurrence interval (years)	Imperviousness I (percent)	Flood-peak-magnitude ratio: Completely sewered basin having alined channels and specified $L/\sqrt{S}$ value			Basin having sewered tributaries, unalined main channels, and specified $L/\sqrt{S}$ value		
		0.1	1	10	0.1	1	10
2.33	0	3.07	2.76	2.44	2.40	2.20	2.00
	20	3.99	3.59	3.17	3.11	2.86	2.60
	50	5.37	4.83	4.27	4.20	3.85	3.50
	100	7.68	6.90	6.10	6.00	5.50	5.00
25	0	3.07	2.76	2.44	2.40	2.20	2.00
	20	3.27	2.94	2.60	2.56	2.35	2.13
	50	3.64	3.28	2.90	2.85	2.61	2.38
	100	4.12	3.70	3.27	3.22	2.95	2.68
50	0	3.07	2.76	2.44	2.40	2.20	2.00
	20	3.12	2.81	2.48	2.44	2.24	2.03
	50	3.30	2.96	2.62	2.58	2.36	2.15
	100	3.49	3.13	2.77	2.73	2.50	2.77
100	0	3.07	2.76	2.44	2.40	2.20	2.00
	20	3.07	2.76	2.44	2.40	2.20	2.00
	50	3.07	2.76	2.44	2.40	2.20	2.00
	100	3.07	2.76	2.44	2.40	2.20	2.00

Table 5 supports the following general conclusions:

1. The effect of sewer installation, independent of impervious development, is to increase flood-peak magnitudes by a factor of two to three.
2. A completely impervious surface increases the average-size flood by a factor of 2½, but impervious surface has a decreasing effect upon larger floods and has an insignificant effect upon the 100-year flood.
3. For the type of suburban development expected in the study area, flood peaks on most basins will be increased by a factor of two to three.

USE OF RELATIONS

Relations defined in the analysis may be combined and summarized as

$$Q_x=\bar{Q}R=230KRA^{0.82}T^{-0.48}. \qquad (4)$$

where Q_x is the magnitude of a flood of x-year recurrence interval, in cubic feet per second, $\bar{Q}$ is the magnitude of the average flood, in cubic feet per second, R is the flood-frequency ratio determined from figure 5, K is the coefficient of imperviousness computed from equation 1, A is basin area, in square miles, and T is lag time, in hours, determined from figure 4.

By use of appropriate values of K, T, A, and R, equation 4 may be used to estimate the future flood magnitude for any recurrence interval from 2.33 to 100 years, for either natural or developed conditions. Three physical characteristics of the basin, area, length, and slope, are required and may generally be obtained from topographic maps. Percentage of manmade impervious surface in the basin may be measured from aerial photographs but in actual practice will probably be estimated for the degree of development expected.

The following is an example of the application of the presented relations: Suppose it is desired to estimate the 25-year-peak discharge on Rabbit Branch near Burke, Va., for an expected future development consisting of a 40-percent-impervious surface and a drainage class defined as developed basin, having unalined main channels (class *B*). From topographic maps measure

A=3.81 square miles;

L=0.34 mile—10 percent of the distance to the basin rim,
=2.9 miles—85 percent of the distance to the basin rim,
=3.4 miles to the basin rim;

Elevation=282 feet at 10 percent of the distance to the basin rim,
=395 feet at 85 percent of the distance to the basin rim.

Compute

$$S=\frac{395-282}{2.90-0.34}=\frac{113}{2.56}=44.2 \text{ ft/per mi};$$

$$T=0.9\ (L/\sqrt{S})^{0.50}=0.9\left(\frac{3.4}{\sqrt{44.2}}\right)^{0.50}=0.64 \text{ hours};$$

K=1.60 for a 40-percent-impervious surface;

R=2.34 for a 40-percent-impervious surface and 25-year recurrence interval from figure 5;

$$Q_{d_{25}}=230\times 1.60\times 2.34\times (3.81)^{0.82}\times (0.64)^{-0.48}=3{,}200 \text{ cubic feet per second.}$$

SUMMARY

Relations have been presented for estimating the magnitude of flood peaks for recurrence intervals having a maximum of 100 years on drainage basins having various degrees of urban or suburban development. Drainage area, length, and slope of the basin are required to use the relations. Information on the type of drainage-system improvement and the extent of impervious basin surface is also required, but in actual practice this information will usually be available from master plans depicting probable future development.

The relations were defined using all available information about the study area and pertinent data collected in other areas. They are considered most applicable to basins in the Washington metropolitan area which are in a rural condition or are in a completely developed urban or suburban condition.

The lag time appears to be the basin characteristic most affected by urbanization. For the streams studied in this report, the lag time for a completely storm-sewered system is about one-eighth that of a com-

parable natural system, while storm sewering of only the tributaries (main channels unalined) reduces the lag time to about one-fifth that of a comparable natural system.

The relations indicate that urban and suburban development significantly changes flood magnitudes. On small, steep basins, drainage improvements alone may triple average-flood magnitudes, and complete development of stream channels with a 100-percent-impervious cover may increase average floods by a factor of nearly eight. The type of development anticipated in the city of Alexandria and Fairfax County can be expected at least to double the magnitude of all floods.

Information presented in this report should be an aid to engineers and planners who are charged with guiding future land use and development.

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