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Flood Frequencies and Bridge and Culvert Sizes for Forested Mountains of North Carolina

by

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INTRODUCTION

Executive Order 11296, issued in August 1966, expresses the Federal Government's concern over mounting losses of lives and property as a result of floods in the United States. This order requires all Federal executive agencies to evaluate flood hazards when planning or constructing Federal facilities, when carrying out programs involving land use planning, and when administering programs supported by Federal funds. At the request of the Office of Management and Budget, guidelines for Federal agencies were developed and published by the U.S. Water Resources Council (1972). Although headwater areas are recognized, these guidelines are oriented primarily toward problems in the flood plains, where risk to life and damage to property may occur from overflow of streams. However, guidelines are also needed for headwater areas, where private individuals and State and Federal agencies must estimate flood flows in order to design culverts, bridges, and other structures.

Because of their simplicity, empirical formulas have been widely used to estimate discharge. Chow (1962) reviewed over 100 empirical or semiempirical formulas for estimation of flood flow. He found that in 1852 John Roe had prepared a drainage table of sewer sizes and slopes for the city of London. In this country, Major E. T. C. Meyers was one of the first engineers to propose a formula for determining waterway area. Chow also surveyed all State Highway Departments and found that 58 percent of those who responded used A. N. Talbot's 1887 formula, with or without modification, for determining waterway area. Use of empirical formulas has limitations and disadvantages. Many such formulas were derived for specific areas and conditions and cannot be applied to other areas or conditions. Formulas such as Talbot's often contain a coefficient to adjust the basic equation for local conditions, but selection of the proper coefficient requires testing and good judgment. Furthermore, the probability of the recurrence of floods of given sizes is often disregarded in empirical formulas.

Because knowledge of expected flow discharges from small forested watersheds is needed for realistic design of culverts and bridges, a study was made of the recurrence interval of flood flows from forested lands in the Blue Ridge Province of North Carolina. Relationships between discharge at recurrence intervals and drainage area and elevation are presented in this paper. Capacity tables for several types and sizes of culverts are also presented to simplify problems in culvert design for both the engineer and nonengineer.

DESCRIPTION OF AREA

Data used in this analysis came from two sources: experimental forested watersheds, approximately 0.1 to 3 sq. mi. in size, operated by the Forest Service at the Coweeta Hydrologic Laboratory near Franklin, North Carolina; and predominantly forested watersheds, approximately 13 to 50 sq. mi. in size, located west of Asheville, North Carolina, and gaged by U. S. Geological Survey. All of these watersheds lie within the Blue Ridge Province of the Appalachian Mountain Physiographic Division. Precipitation of the region is distributed fairly evenly throughout the year, and snow constitutes only about 5 percent of the total. Annual rainfall totals about 50 inches in the Asheville vicinity and increases generally with elevation and in a southwesterly direction from Asheville. The depth-duration of rainstorms also follows a similar pattern. The Coweeta Hydrologic Laboratory lies within one of the regions with the highest rainfall in the East, with rainfall at the Laboratory varying from an average of 70 inches at 2,200 ft. to approximately 100 inches at 5,000 ft. Rainfall from convection storms predominates during the summer months, whereas precipitation during the dormant season is usually associated with frontal activity. Hurricanes occasionally influence rainfall in the mountains, and flood flows may originate from convection, frontal, or hurricane sources.

Elevations of all gaging stations in the study are 2,200 ft. or greater. Maximum elevations of individual watersheds range from about 3,000 ft. for small watersheds within the Coweeta basin to over 6,000 ft. for some of the larger watersheds gaged by U. S. Geological Survey. Carolina gneiss of Pre-Cambrian origin forms a basement rock of granite gneiss, mica gneiss, and mica schist. The basement complex includes, in places, a thick series of late Pre-Cambrian sedimentary rock which has undergone metamorphosis. The weathered solum on slopes at lower elevations is often 50 or more feet deep, whereas soils are immature and shallow at elevations greater than 4,500 ft. Rock outcrops are fairly common above 5,000 ft. Soils which have developed under hardwood forests usually have infiltration rates greatly exceeding the maximum rate of rainfall observed in the area; therefore, overland flow is uncommon and most water reaches streams by subsurface flow. The drainage pattern is dendritic in shape, and stream density is high--often 10 mi. of streams occur per square mile of land area. Steeply sloping mountains, humid climate, and deep soils all combine to give relatively stable, year-round flows.

METHODS

Watersheds selected at Coweeta were covered with hardwoods and were either undisturbed or previously treated. If the watershed had some vegetative treatment in the past, the period when runoff would have been affected was deleted from the record and peak flows were estimated by the methods suggested by Dalrymple (1960).

Five Geological Survey watersheds in the same physiographic region were also suitable and selected for use. These watersheds were between 13 and 52 sq. mi. in size, had no significant urban or agricultural development, i.e., they were primarily forested, and contained no major impoundments or diversions of water which would materially affect peak discharge. Some physical and other characteristics of the watersheds are listed in table 1.

The flow-frequency methods described by Dalrymple (1960) were used to determine the recurrence interval of flood flows, to test the watershed data for homogeneity, and to formulate the relationship between discharge and drainage area.

The recurrence interval of each maximum discharge from the various watersheds was calculated by the formula

$$T = \frac{n + 1}{m} \tag{1}$$

where T is the recurrence interval in years, i.e., the average interval of time within which the magnitude of the event will be equaled or exceeded once, n is the number of years of record, and m is the magnitude of flood, the highest being 1. Discharge was plotted over recurrence interval for each storm. Instead of a mathematically fitted line, a straight line was fitted to the data by eye. The graphical mean has been found to be more stable and dependable than the arithmetic mean because, with graphical means, greater weight is given to medium-sized floods than to extreme floods. Furthermore, the graphical mean is not adversely influenced by the chance inclusion or omission of a major flood. Care was taken to assure a good fit near the 2.33-year recurrence period, which is defined as the mean annual flood.

RESULTS

Figure 1 illustrates the relationship between peak discharge and recurrence interval for Coweeta Watershed 8 (2.9 sq. mi.) and for the Davidson River drainage (40 sq. mi.). In general, the relationship for smaller watersheds was a straight line when plotted on log-log paper; however, at recurrence intervals below 1.5 years, the data departed from the linear form and discharge decreased sharply as the recurrence interval dropped from 1.5 to 1.0 year. In contrast, the data for large watersheds plotted as curves on log-log paper, but they also showed decline in discharge at the lower recurrence intervals.

Table 1.--Physical and other characteristics of the watersheds used in the study of flow frequency in the Blue Ridge Province of North Carolina

Watershed[1]	Size	Elevation		Ratio[2] $\frac{Q_{10}}{Q_{2.33}}$	Adjusted years of record
		Maximum	Minimum		
	Sq. mi.	---- Ft. ----			No.
Coweeta					
1	0.0625	3,241	2,313	1.92	27
2	.0468	3,314	2,327	1.79	36
8	2.932	5,252	2,302	1.67	36
10	.3312	3,885	2,436	1.62	27
14	.2359	3,125	2,318	1.65	34
18	.0484	3,320	2,382	1.89	34
19	.1093	3,650	2,440	1.76	27
21	.0937	3,853	2,700	1.49	31
22	.1328	4,081	2,780	1.74	23
27	.1500	4,785	3,380	1.66	30
28	.5560	5,087	3,162	1.70	26
32	.1593	4,100	3,020	1.65	29
34	.1265	3,960	2,783	1.64	30
36	.1875	5,052	3,350	1.82	28
37	.1687	5,252	3,390	1.64	24
Davidson River	40.4	5,960	2,115	1.95	51
West Fork of Pigeon River	27.6	6,410	2,976	1.95	18
East Fork of Pigeon River	51.5	6,214	2,674	2.04	18
Nantahala River	51.9	5,499	3,073	1.72	31
Noland Creek	13.8	6,642	2,280	1.58	36

[1]Coweeta watersheds were completely forested; those gaged by U. S. Geological Survey were at least 90 percent forested.

[2]$\frac{\text{Discharge (Q) at recurrence interval of 10 years}}{\text{Discharge (Q) at recurrence interval of 2.33 years}}$.

A homogeneity test was conducted to ensure that data from the small experimental watersheds and the much larger and partially forested watersheds could be combined. This test is discussed by Dalrymple (1960) and is based on the Gumbel distribution. In the test, the discharges (Q) at recurrence intervals of 10 years and 2.33 years were used. The average ratio of $Q_{10}/Q_{2.33}$ was calculated and then multiplied by the discharge of the mean annual flood of each watershed.

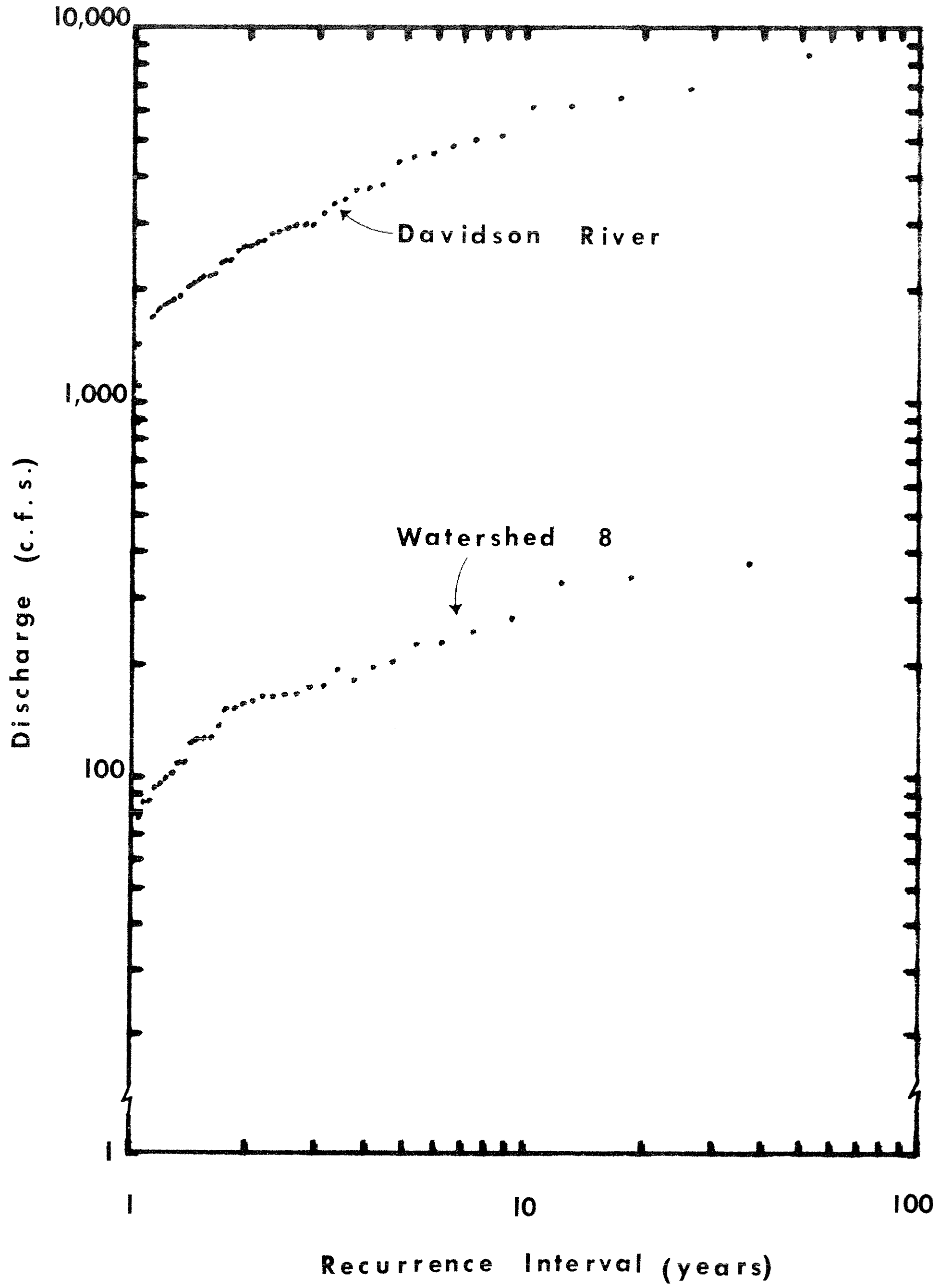

Figure 1.--Discharge vs. recurrence interval for Coweeta Watershed 8 (2.9 sq. mi.) and the Davidson River drainage (40 sq. mi.).

The recurrence interval for these discharges was then determined. The recurrence intervals are shown in figure 2, along with the upper and lower confidence limits computed for the 95-percent confidence interval.

Because the data for both large and small watersheds plotted within the confidence interval, the data could be combined into a regional frequency curve. However, the relationship of the mean annual flood on drainage area in square miles (fig. 3) showed considerable scatter, and

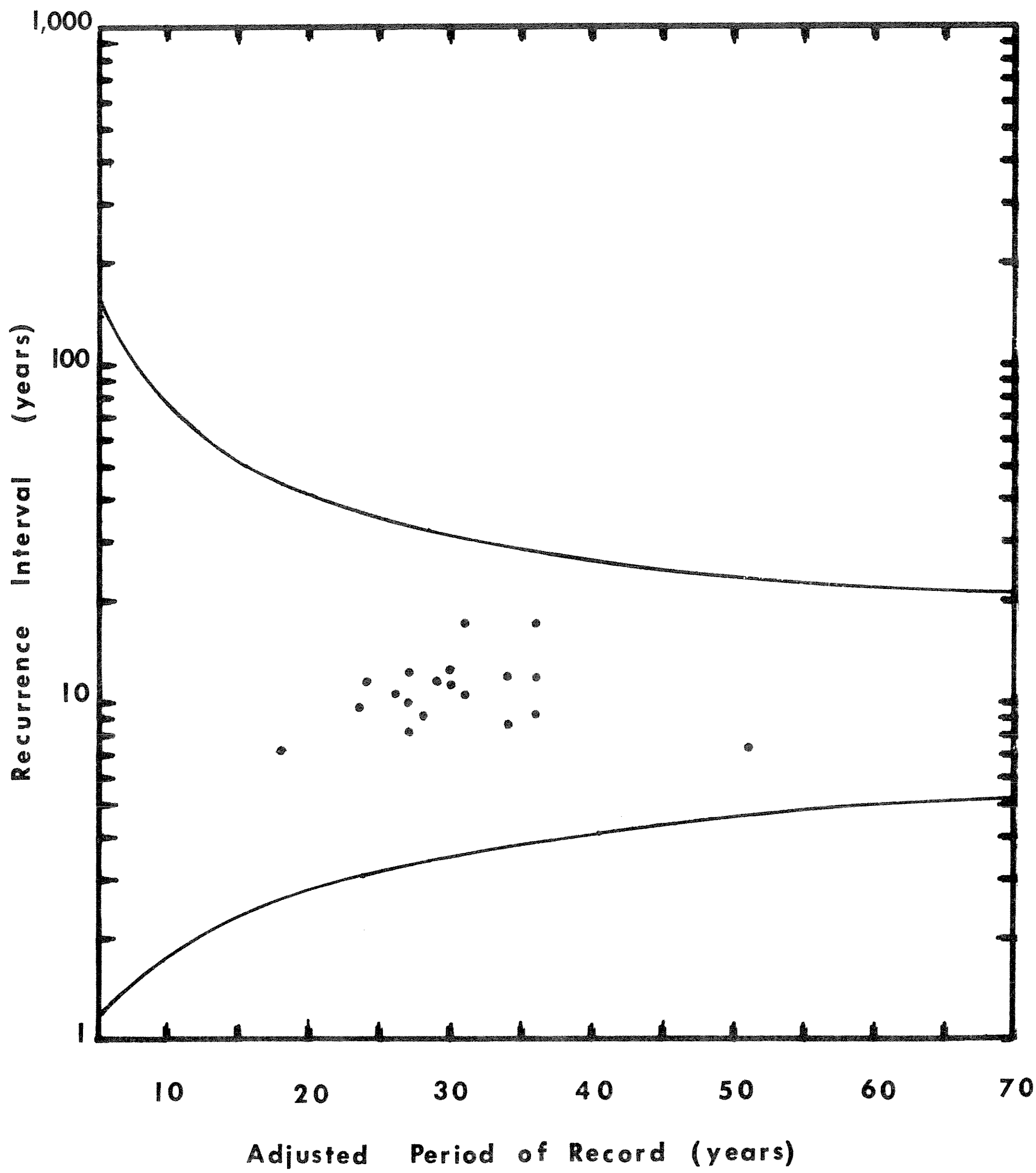

Figure 2.--Homogeneity of data for discharges at recurrence intervals of 10 years from both large and small watersheds. (The upper and lower lines are the 95-percent confidence limits.)

this scatter suggested that there is a major source of variation other than drainage area. Differences in slope, a decrease in soil depth with elevation, and an increase in precipitation with elevation for the area studied all suggested that some measure of the elevation of the watershed, either at the control, at midelevation, or at the maximum elevation along the ridge line, might integrate these effects and reduce the variation in flow between watersheds. Area and elevation are useful parameters in estimating discharge because both are easily determined from the 1:24,000-scale topographic maps of western North Carolina issued by the U.S. Army Corps of Engineers. The model which relates these variables to discharge is

$$Q_i = aA^bE^c \tag{2}$$

where Q_i is discharge in cubic feet per second for recurrence interval i, A is drainage area in acres, E is elevation of the watershed in feet, and a, b, and c are coefficients.

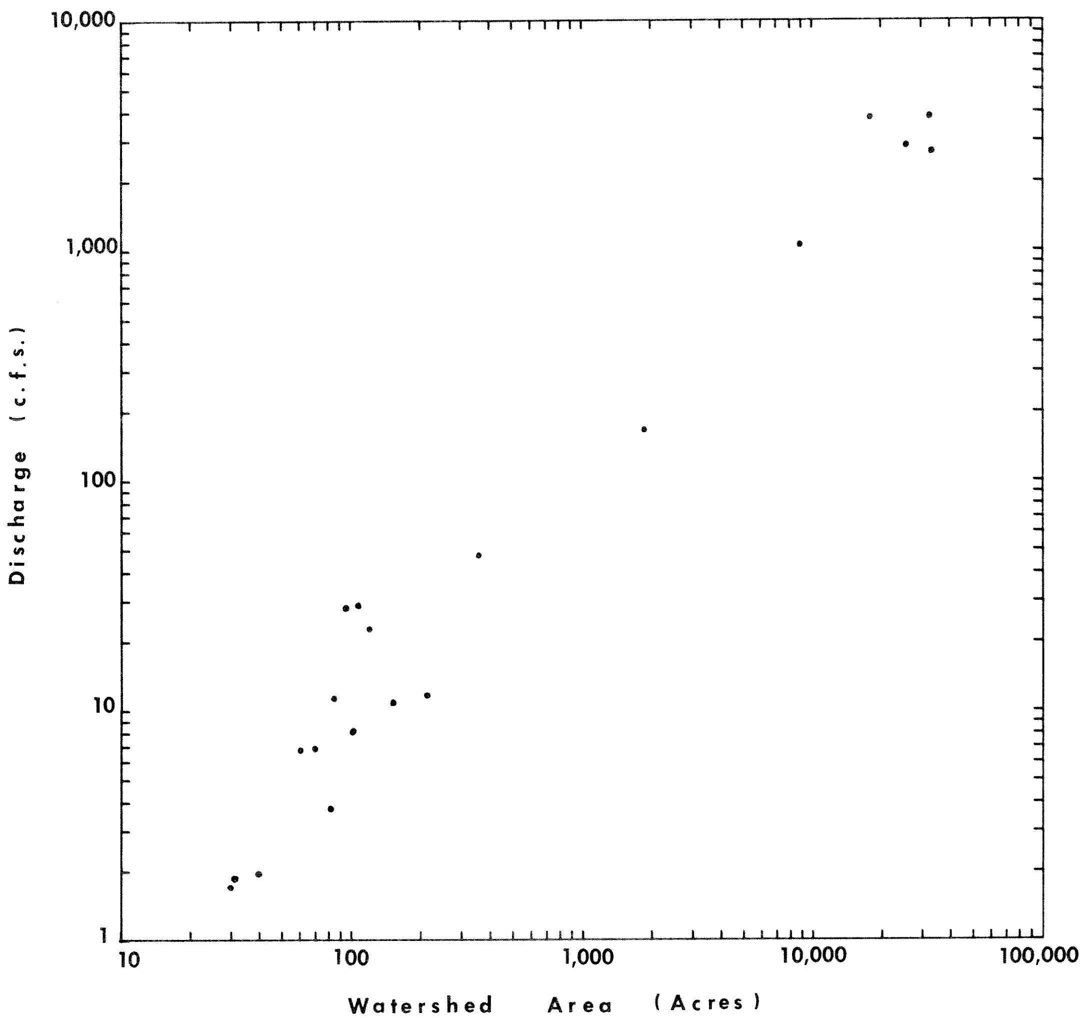

Figure 3.--Relationship between the mean annual discharge (2.33-year recurrence interval) and watershed area for both large and small watersheds.

The multiple regression in which both area and elevation were used was a significant improvement over the regression in which only drainage area was used as the independent variable. Elevation of the control and midelevation of the watershed both worked well, but they are more difficult to determine than the maximum elevation of the watershed. Therefore, elevation of the highest point on the watershed boundary was selected as the second independent variable. The r^2's for the equations were all greater than 0.98. Thus, 98 percent of the variation in discharge is explained by these equations when drainage area and maximum elevation are used. Any additional variables will not increase reliability significantly and will only serve to make these equations more difficult to apply. The derived equations for the 20 watersheds studied and their r^2's are listed below:

Equation	r^2	
$\text{Log } Q_{2.33} = -11.580 + 0.803 (\log A) + 3.038 (\log E)$	0.985	(3)
$\text{Log } Q_{5} = -11.298 + 0.819 (\log A) + 2.986 (\log E)$	0.984	(4)
$\text{Log } Q_{10} = -10.962 + 0.823 (\log A) + 2.920 (\log E)$	0.984	(5)
$\text{Log } Q_{20} = -10.727 + 0.820 (\log A) + 2.887 (\log E)$	0.983	(6)
$\text{Log } Q_{30} = -10.509 + 0.816 (\log A) + 2.846 (\log E)$	0.982	(7)
$\text{Log } Q_{40} = -10.461 + 0.810 (\log A) + 2.848 (\log E)$	0.981	(8)
$\text{Log } Q_{50} = -10.575 + 0.804 (\log A) + 2.893 (\log E)$	0.983	(9)

Figures 4 and 5 show the relationship between drainage area and elevation of the highest point on the watershed boundary for floods at recurrence intervals at 20 and 50 years. Comparison of discharges for the same elevation and drainage area indicates that discharge for the 50-year flood is approximately 1.4 times greater than the discharge for the 20-year flood. In comparison, an increase of 500 ft. in watershed elevation for floods of the same recurrence interval increases discharge by a factor of about 1.3 to 1.6. This relationship demonstrates that elevation of the catchment is critical in estimating peak discharge in the North Carolina mountains.

Also plotted in figure 5 is the discharge predicted by Talbot's formula for calculating waterway area in the mountains if a velocity of flood flow of 6 ft./sec. is assumed. Designing waterway areas by Talbot's formula would result in substantial overdesign for the 50-year flood, even for a watershed with a maximum elevation of 6,500 ft. The overdesign would be progressively greater as maximum elevation of the watershed decreased or as velocity of flood flow increased.

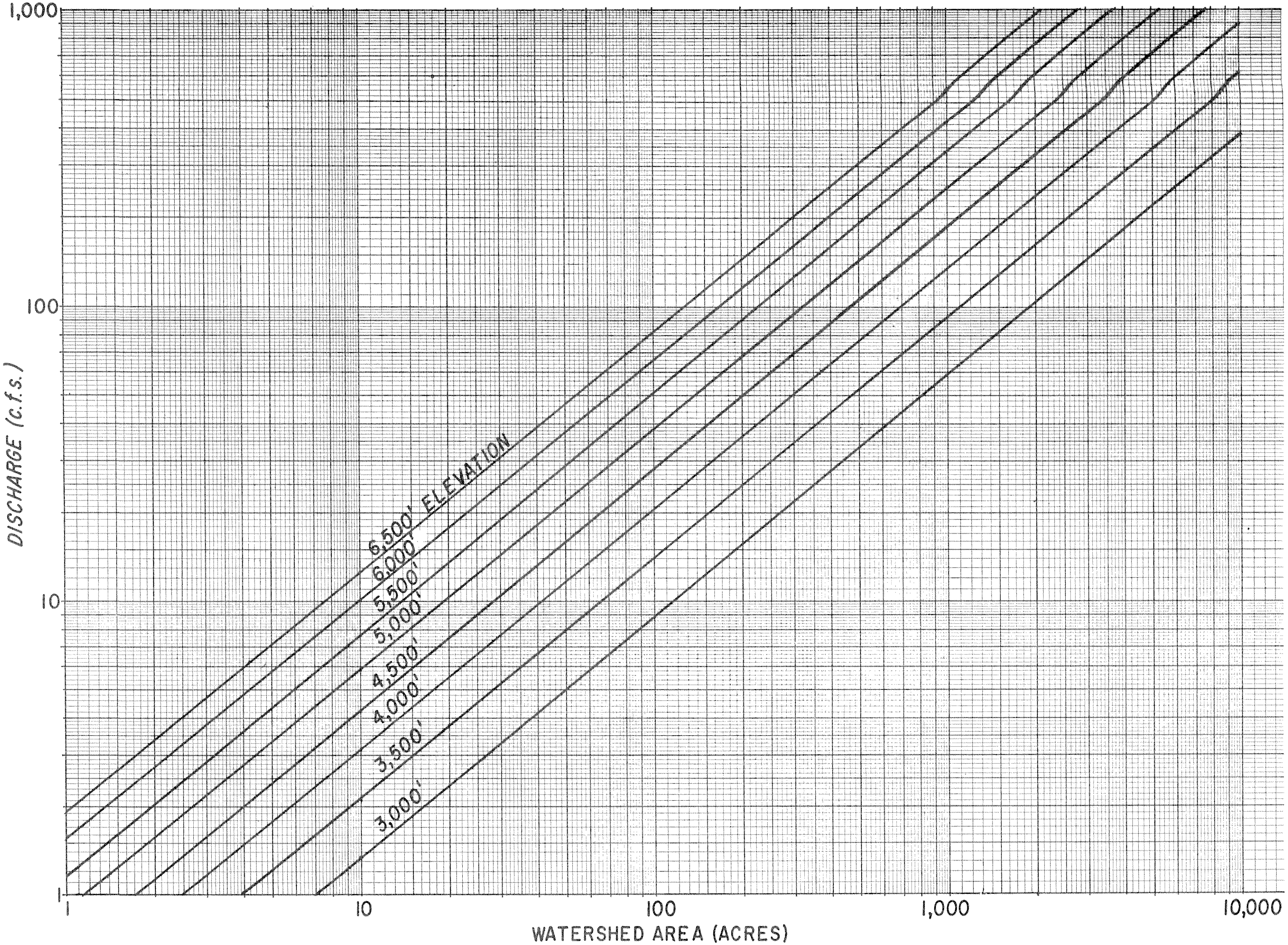

Figure 4.--Discharge for floods at recurrence intervals of 20 years according to drainage area and elevation of the watershed at its highest point.

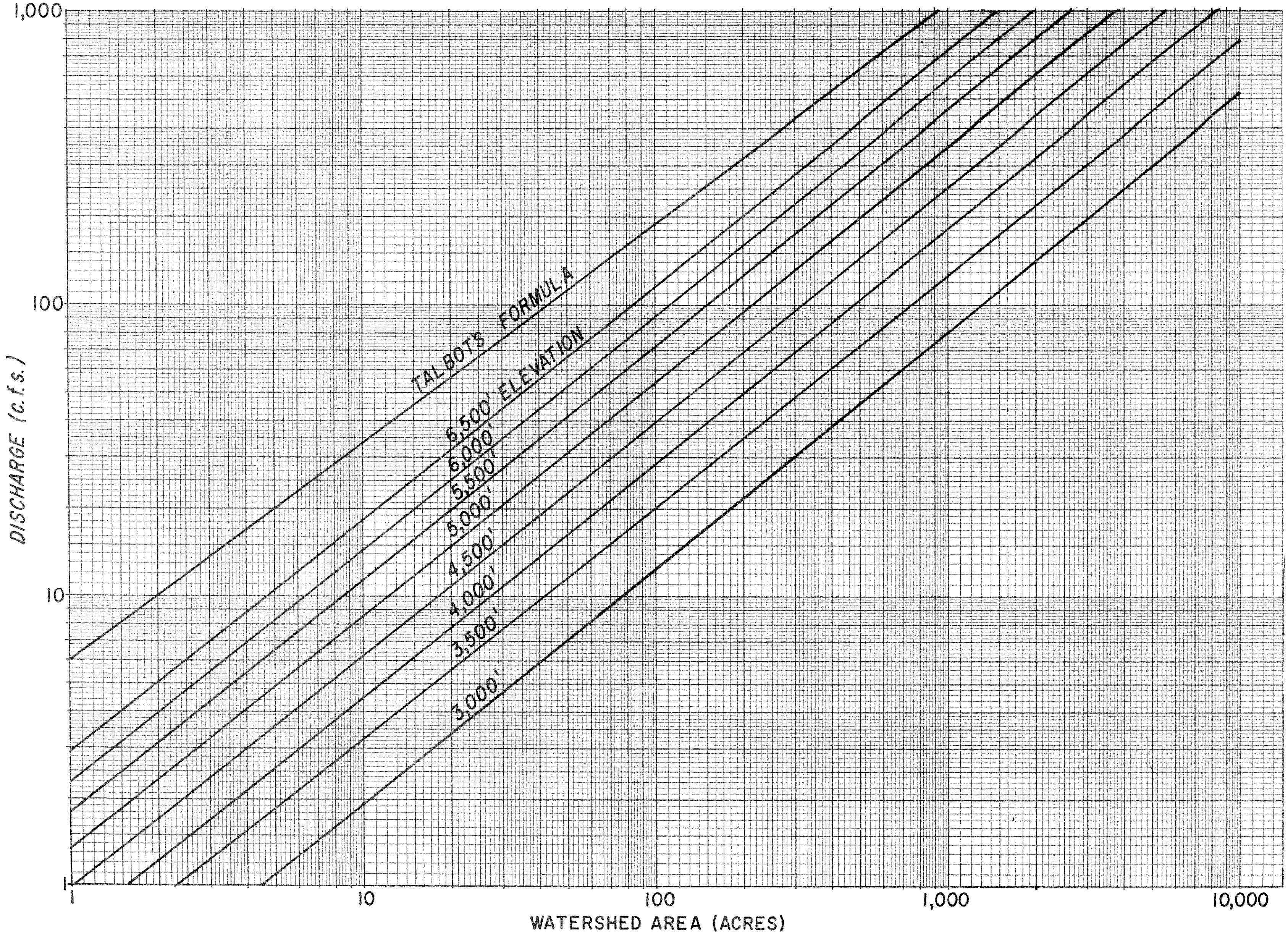

Figure 5.--Discharge for floods at recurrence intervals of 50 years according to drainage area and elevation of the watershed at its highest point. Talbot's formula for waterway area through which the flow must pass is C $\sqrt[4]{\text{acres}^3}$ where C is a coefficient. When C is set at 1.0 for steep mountain land and the velocity of flow is set at 6 ft./sec., the discharge by Talbot's formula is estimated as 6 (1) $\sqrt[4]{\text{acres}^3}$.

Geological Survey conducted an analysis of flood frequency for North Carolina in which areas were designated according to their response to rainfall and their peak discharges (Forrest and Speer 1961). Of the watersheds gaged by Geological Survey, those used in my study fall within hydrologic areas 1 and 2 as designated by Forrest and Speer. When equation (3) was solved for area and maximum elevation, four of these five watersheds fell closest to their assigned hydrologic area. The slope of equation (3), however, is somewhat greater than that presented for hydrologic areas 1 and 2. This disparity suggests that perhaps the slope of the relationship between discharge and drainage area changes as watershed area decreases to small-sized drainages. Thus, considerable error may be introduced if the Geological Survey curves for hydrologic areas are extrapolated to smaller-sized drainage than indicated or if equation (3) is extrapolated to watersheds larger than 50 sq. mi. Because the data used to derive equations (3) through (9) were for watersheds with maximum elevations of 3,000 ft. or greater, the equations are not applicable for elevations below 3,000 ft. The equations also should not be applied to areas outside the Blue Ridge Province of North Carolina or to areas larger than 50 sq. mi. until the applicability of the equations to those areas has been determined.

APPLICATION TO CULVERT AND BRIDGE DESIGN

Once the discharge of a drainage can be estimated with reliability, the next problem in design is to determine the waterway area to carry this flow. In bridge design, the structure must be able to pass the discharge for which it was designed without damage to the structure or the channel downstream. Three factors are of primary concern: designed discharge (Q), which can be obtained from the equations or graphs presented; waterway area through which the flow must pass (A); and velocity of flow (V). The basic equation by Chezy for determining flow of water in open channels is written as follows:

$$Q = AV \tag{10}$$

or, solving for waterway area:

$$A = Q/V \tag{11}$$

A measure of velocity can be obtained by solving Manning's formula (American Iron and Steel Institute 1967; Spindler 1958), which considers slope, wetted perimeter, and roughness of the channel. A second and preferable approach is to use the actual stream velocity determined for major floods in the area in question. For mountain streams, measured channel velocity varies from about 10 to 25 ft./sec. for major floods on large watersheds (Tennessee Valley Authority 1961, 1963, 1964); in the smaller watersheds such as those at Coweeta, velocity for the 20- to 50-year flood is much lower, ranging from about 5 to 7 ft./sec. If the lower velocity for floods at recurrence intervals of 20 years or greater is used, equation (11) can be solved to obtain a conservative estimate of the waterway area required to pass flood peaks. For example, discharge for the 50-year flood from a 640-acre watershed with a maximum elevation of 3,000 ft. is about 55 c.f.s. (from equation (9) or figure 5). If an average velocity of 5 ft./sec. is assumed, a waterway area of 11 sq. ft. would be required to carry this flow.

Selection of the proper culvert size for designed discharge is much more difficult. There is no unique solution for a particular size or type of culvert. The required culvert size depends upon whether flow through the culvert is controlled at the inlet or outlet. Inlet control means that the culvert discharge is controlled at the entrance by depth of headwater (HW), entrance geometry including culvert shape and cross-sectional area, and the type of inlet edge (fig. 6). In outlet control, slope (S), Length (L), and roughness of the culvert are additional considerations.

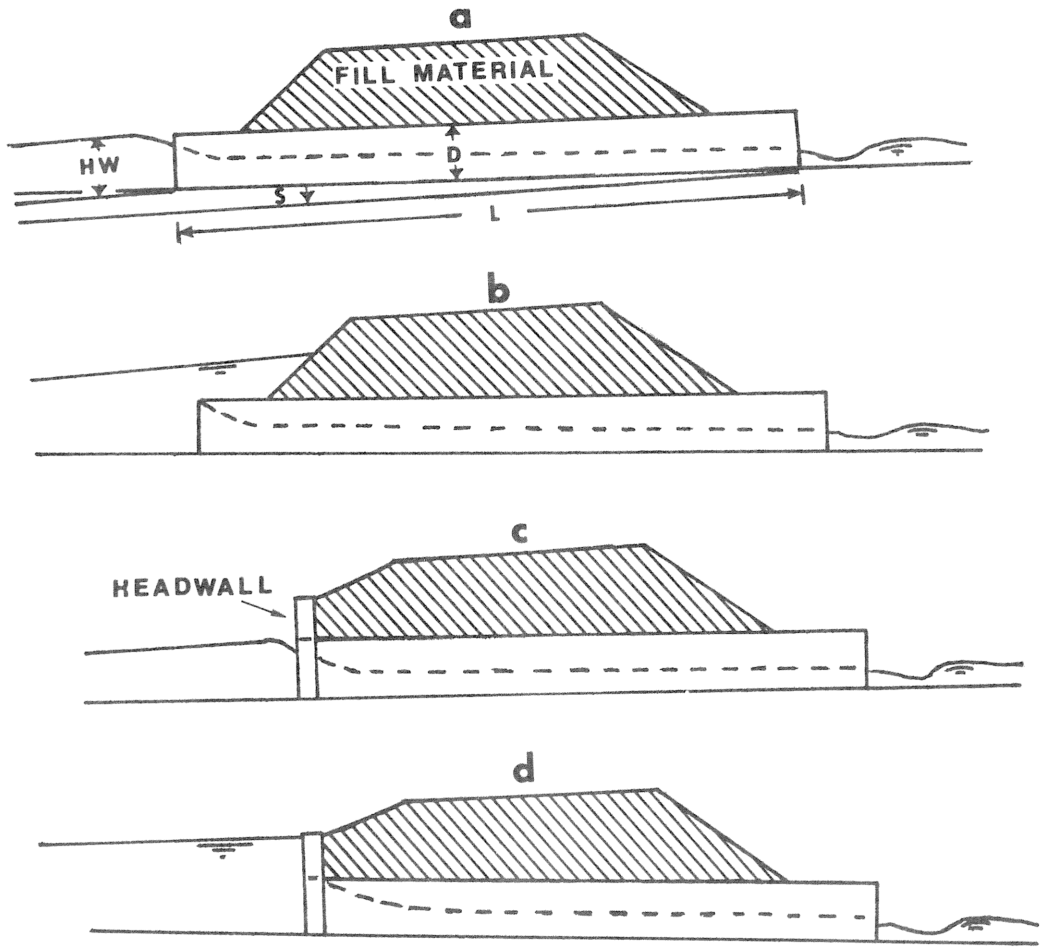

Figure 6.--Inlet control for two types of culverts and two depths of headwater. Schematics a and b illustrate culverts which project into the inlet basin. In a, headwater depth (HW) is equal to the culvert diameter (D) or rise of pipe-arch culverts; in b, headwater depth is greater than culvert diameter. The two examples of headwater depth are shown for culverts with headwalls in c and d. Other dimensions are culvert length (L) and slope (S).

The Federal Highway Administration recently developed relationships between headwater and discharge for various types of circular and pipe-arch culverts. These relationships, which are based on laboratory models, prototype testing, and experimental data, are presented as a series of nomographs. For a more complete discussion of inlet and outlet control and for the nomographs, Hydraulic Engineering Circulars 5 and 10 should be consulted (Herr and Bossy 1965a, 1965b).

Fortunately, for most situations in mountainous streams, inlet control will prevail. Culvert lengths will often be short, slopes will be relatively steep, and length and barrel slope can be ignored. Because most natural channels are wide in comparison with culvert diameter, the depth of water in the natural channel is considerably less than the critical depth of water in the pipe; thus, critical depth will not be a limitation. Unless the channel is restricted below the culvert by terrain features, debris, or protruding rocks (which will reduce the velocity of water discharging from the culvert), inlet control can usually be assumed for mountain streams.

As the headwater depth at the entrance of the culvert increases, the discharge capacity of the culvert increases. For design purposes, the minimum headwater depth should be set equal to the height (or diameter) of the culvert. For landowners who do not know the exact depth of fill which will cover the culvert, culvert size should be selected so that the ratio of headwater depth to culvert diameter will equal unity. In this situation, the culvert will not flow completely full (pipe flow), but more important, the water level in the inlet basin will not exceed the culvert height predetermined for the designed discharge. If engineers design for HW/D ratios greater than 1, they can take advantage of the greater discharge capacity of culverts. For example, the culvert capacity when the HW/D ratio is 2.0 is approximately twice that of the same culvert diameter when the HW/D ratio is 1. Thus, a smaller (and less expensive) culvert can be selected if the engineer will permit water to pond occasionally against the fill above the top of the culvert.

To aid the landowner or engineer in selecting appropriate culvert diameters, capacity tables (tables 2, 3, 4, and 5) were developed for inlet control on the basis of Hydraulic Engineering Circulars 5 and 10. Because friction in long culverts and low culvert slopes reduce culvert capacity and may change control from inlet to outlet, these tables contain a guide to ratios of length to slope (L/100S) for each culvert diameter. As long as these L/100S ratios are not exceeded and downstream flow is not restricted by terrain, debris, or rocks, inlet control may be assumed for mountainous conditions. When these conditions are not met, the tables should not be used and Hydraulic Engineering Circulars 5 and 10 should be consulted.

Table 2.--Capacity of standard circular pipes of corrugated metal with projecting end and inlet control[1]

Pipe diameter (inches)	Ratio[2] L/100S	Capacity of pipe with HW/D ratio[3] of--										
		1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
		c.f.s.										
12	5	2.1	2.3	2.6	2.7	2.9	3.1	3.3	3.6	3.8	4.0	4.2
15	5	3.7	4.1	4.4	4.8	5.1	5.5	5.8	6.2	6.6	6.9	7.2
18	10	5.8	6.4	7.0	7.5	8.1	8.7	9.2	9.8	10.3	10.9	11.4
21	15	8.6	9.4	10.2	11.1	11.9	12.7	13.5	14.4	15.2	16.0	16.8
24	20	12.0	13.1	14.3	15.4	16.6	17.7	18.9	20.0	21.2	22.3	23.5
27	25	16.1	17.6	19.2	20.7	22.3	23.8	25.4	26.9	28.4	30.0	31.5
30	30	20.9	22.9	24.9	27.0	29.0	31.0	33.0	35.0	37.0	39.0	41.0
33	40	26.5	29.1	31.6	34.2	36.7	39.3	41.8	44.4	46.9	49.5	52.0
36	50	32.9	36.1	39.3	42.5	45.7	48.8	52.0	55.1	58.3	61.4	64.6
42	60	48.4	53.1	57.8	62.4	67.1	71.8	76.4	81.0	85.7	90.3	94.9
48	80	67.5	74.1	80.6	87.1	93.6	100.1	106.6	113.1	119.6	126.0	132.5
54	100	90.6	99.4	108.2	116.9	125.6	134.4	143.1	151.8	160.4	169.1	177.8
60	130	117.8	129.3	140.7	152.1	163.5	174.8	186.1	197.4	208.7	220.0	231.2
66	150	149.5	164.0	178.5	193.0	207.4	221.8	236.1	250.5	264.8	279.1	293.4
72	170	185.8	203.8	221.8	239.8	257.7	275.6	293.4	311.2	329.0	346.8	364.5
78	200	226.8	248.9	270.9	292.8	314.7	336.5	358.3	380.1	401.8	423.5	445.2
84	230	273.0	299.5	326.0	352.3	378.7	404.9	431.2	457.4	483.5	509.6	535.7
90	265	324.4	355.8	387.2	418.6	450.0	481.1	512.2	543.3	574.4	605.4	636.4
96	300	381.0	418.0	454.9	491.7	528.5	565.2	601.8	638.3	674.8	711.2	747.6

[1]Based on Hydraulic Engineering Circulars 5 and 10 (Herr and Bossy 1965a, 1965b).
[2]L = length of culvert; S = slope of culvert.
[3]HW = headwater depth; D = diameter of culvert.

Table 3.--Capacity of standard circular pipes of corrugated metal with headwalls and inlet control[1]

Pipe diameter (inches)	Ratio[2] L/100S	Capacity of pipe with HW/D ratio[3] of--										
		1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
		c.f.s.										
12	5	2.4	2.6	2.8	3.1	3.3	3.6	3.8	4.0	4.3	4.5	4.8
15	5	4.1	4.5	4.9	5.4	5.8	6.2	6.6	7.1	7.5	7.9	8.3
18	10	6.5	7.1	7.8	8.5	9.1	9.8	10.5	11.1	11.8	12.5	13.1
21	15	9.5	10.5	11.5	12.4	13.4	14.4	15.4	16.4	17.3	18.3	19.3
24	20	13.3	14.6	16.0	17.4	18.7	20.1	21.5	22.8	24.2	25.6	26.9
27	25	17.8	19.6	21.5	23.3	25.1	27.0	28.8	30.6	32.5	34.3	36.1
30	30	23.2	25.6	27.9	30.3	32.7	35.1	37.4	39.8	42.2	44.6	47.0
33	40	29.4	32.4	35.4	38.4	41.5	44.5	47.5	50.5	53.6	56.6	59.6
36	50	36.5	40.3	44.0	47.8	51.5	55.3	59.0	62.8	66.6	70.3	74.1
42	60	53.7	59.2	64.7	70.2	75.7	81.2	86.7	92.2	97.8	103.3	108.9
48	80	74.9	82.6	90.2	97.9	105.6	113.3	121.0	128.7	136.5	144.2	151.9
54	100	100.5	110.8	121.1	131.4	141.7	152.0	162.4	172.7	183.1	193.5	203.9
60	130	130.8	144.1	157.5	170.9	184.3	197.7	211.2	224.7	238.2	251.7	265.2
66	150	165.9	182.8	199.8	216.8	233.8	250.8	267.9	285.0	302.1	319.2	336.4
72	170	206.1	227.1	248.2	269.3	290.5	311.7	332.9	354.1	375.4	396.6	418.0
78	200	251.7	277.4	303.1	328.9	354.7	380.6	406.5	432.4	458.4	484.3	510.4
84	230	302.8	333.7	364.7	395.7	426.8	452.9	489.0	520.2	551.5	582.7	614.0
90	265	359.7	396.4	433.2	470.0	506.9	543.9	580.9	618.0	655.1	692.2	729.4
96	300	422.5	465.7	508.9	552.2	595.5	639.0	682.4	726.0	769.6	813.2	856.9

[1]Based on Hydraulic Engineering Circulars 5 and 10 (Herr and Bossy 1965a, 1965b).
[2]L = length of culvert; S = slope of culvert.
[3]HW = headwater depth; D = diameter of culvert.

Table 4.--Capacity of standard pipe-arches of corrugated metal with projecting end or headwalls and inlet control[1]

Pipe dimensions (inches)	Ratio[2] L/100S	Capacity of pipe-arch with HW/D ratio[3] of--										
		1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
WITH PROJECTING END												
		c.f.s.										
18 x 11	10	3.0	3.3	3.6	3.9	4.2	4.4	4.7	5.0	5.3	5.5	5.8
22 x 13	15	4.7	5.2	5.6	6.1	6.5	6.9	7.4	7.8	8.2	8.7	9.1
25 x 16	20	6.9	7.6	8.2	8.9	9.5	10.2	10.8	11.4	12.1	12.7	13.3
29 x 18	25	9.6	10.5	11.4	12.3	13.2	14.1	15.0	15.9	16.8	17.6	18.5
36 x 22	35	16.7	18.3	19.9	21.4	23.0	24.5	26.0	27.6	29.1	30.6	32.1
43 x 27	50	26.2	28.7	31.1	33.6	36.0	38.4	40.8	43.2	45.6	48.0	50.4
50 x 31	70	38.4	42.0	45.6	49.1	52.7	56.2	59.7	63.2	66.7	70.2	73.7
58 x 36	90	53.4	58.4	63.4	68.3	73.2	78.2	83.1	87.9	92.8	97.6	102.5
65 x 40	120	71.4	78.1	84.7	91.4	98.0	104.5	111.1	117.6	124.1	130.6	137.0
72 x 44	150	92.6	101.2	109.9	118.5	127.0	135.6	144.1	152.5	160.9	169.3	177.7
WITH HEADWALLS												
18 x 11	10	3.3	3.7	4.0	4.3	4.7	5.0	5.3	5.6	6.0	6.3	6.6
22 x 13	15	5.3	5.8	6.3	6.8	7.3	7.8	8.4	8.9	9.4	9.9	10.4
25 x 16	20	7.7	8.5	9.2	10.0	10.8	11.5	12.3	13.0	13.8	14.5	15.3
29 x 18	25	10.7	11.8	12.8	13.9	14.9	16.0	17.1	18.1	19.2	20.2	21.3
36 x 22	35	18.6	20.5	22.3	24.2	26.0	27.8	29.7	31.5	33.3	35.2	37.0
43 x 27	50	29.3	32.2	33.1	38.0	40.9	43.8	46.6	49.5	52.4	55.3	58.2
50 x 31	70	42.9	47.2	51.4	55.7	59.9	64.1	68.4	72.6	76.8	81.0	85.2
58 x 36	90	59.8	65.7	71.6	77.5	83.4	89.3	95.2	101.1	106.9	112.8	118.7
65 x 40	120	80.1	88.0	95.9	103.8	111.7	119.6	127.5	135.4	143.2	151.1	159.0
72 x 44	150	104.0	114.3	124.5	134.8	145.1	155.3	165.5	175.8	186.0	196.2	206.4

[1] Based on Hydraulic Engineering Circulars 5 and 10 (Herr and Bossy 1965a, 1965b).
[2] L = length of culvert; S = slope of culvert.
[3] HW = headwater depth; D = diameter of culvert.

Table 5.--Capacity of pipe-arches of structural-plate corrugated metal with 18-inch corner radii, projecting end or headwalls, and inlet control[1]

Span size	Rise	Ratio[2] L/100S	Capacity of pipe-arch with HW/D ratio[3] of-- 1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
			- - - -	- - - -	- - - -	- - - -	- - - -	c.f.s.	- - - -	- - - -	- - - -	- - - -	- - - -
WITH PROJECTING END													
6 ft. 1 in.	x 4 ft. 7 in.	40	128	140	152	164	176	188	200	212	224	235	247
7 ft.	x 5 ft. 1 in.	50	174	191	207	224	240	257	273	289	305	322	338
8 ft. 2 in.	x 5 ft. 9 in.	60	250	274	298	322	345	369	392	415	438	462	485
9 ft. 6 in.	x 6 ft. 5 in.	70	341	373	405	438	470	502	533	565	597	628	660
11 ft. 5 in.	x 7 ft. 3 in.	90	477	523	568	613	658	703	747	792	836	880	925
12 ft. 10 in.	x 8 ft. 4 in.	120	683	747	812	877	941	1,005	1,068	1,132	1,195	1,259	1,322
15 ft. 4 in.	x 9 ft. 3 in.	140	924	1,012	1,099	1,186	1,273	1,350	1,446	1,532	1,618	1,703	1,789
16 ft. 7 in.	x 10 ft. 1 in.	170	1,168	1,279	1,390	1,500	1,610	1,719	1,828	1,937	2,046	2,154	2,262
WITH HEADWALLS													
6 ft. 1 in.	x 4 ft. 7 in.	40	143	157	171	185	199	213	227	241	255	269	282
7 ft.	x 5 ft. 1 in.	50	195	214	233	252	271	290	309	328	347	366	385
8 ft. 2 in.	x 5 ft. 9 in.	60	279	307	334	361	389	416	443	470	498	525	552
9 ft. 6 in.	x 6 ft. 5 in.	70	379	417	454	491	528	565	602	639	676	713	750
11 ft. 5 in.	x 7 ft. 3 in.	90	531	583	635	687	739	791	843	894	946	998	1,049
12 ft. 10 in.	x 8 ft. 4 in.	120	757	832	906	980	1,054	1,129	1,202	1,276	1,350	1,424	1,498
15 ft. 4 in.	x 9 ft. 3 in.	140	1,023	1,124	1,224	1,325	1,425	1,525	1,625	1,725	1,825	1,924	2,024
16 ft. 7 in.	x 10 ft. 1 in.	170	1,293	1,420	1,547	1,673	1,800	1,926	2,052	2,178	2,304	2,430	2,556

[1]Based on Hydraulic Engineering Circulars 5 and 10 (Herr and Bossy 1965a, 1965b).
[2]L = length of culvert; S = slope of culvert.
[3]HW = headwater depth; D = diameter of culvert.

Specific Examples

The following examples illustrate the use of the discharge relationships and capacity tables for culverts:

Example 1

Situation: A logging road is constructed to remove timber products intermittently at 10-year intervals for the foreseeable future. This road crosses several intermittent and perennial streams, but damage resulting from overflow of the culverts or bridges will be negligible except at the failure site. Therefore, the landowner decides to set the designed discharge equal to the expected life of the culvert, i.e., the discharge at the recurrence interval of 20 years. At a particular crossing site, the watershed above the site is 56 acres in size and the stream gradient is 10 percent. The maximum elevation of the 56-acre drainage area is 4,600 ft. If a culvert is used, a culvert length of 30 ft. will suffice at the site. The stream channel below the crossing contains no obstructions to flow.

Required: Discharge at the recurrence interval of 20 years, waterway area for a bridge to carry this discharge, and culvert type(s) and sizes which are suitable for use.

From equation (6) or figure 4, the discharge at the recurrence interval of 20 years is estimated to be 19 c.f.s. From equation (11) and an assumed flow velocity of 5 ft./sec., a bridge must provide

$$\frac{19 \text{ c.f.s.}}{5 \text{ ft./sec.}} = 3.8 \text{ sq. ft.}$$

of waterway area.

Because the HW/D ratio is not known in this example, a HW/D ratio of 1.0 should be used. From tables 2 and 3, a circular culvert of corrugated metal 30 inches in diameter with projecting end or headwalls would carry the designed discharge of 19 c.f.s. From table 4, a 43- by 27-inch pipe-arch with projecting end or headwalls would also carry the designed discharge. Applicability of the capacity tables for inlet control can be tested by calculating the L/100S ratio: 30/10 = 3, which is less than the L/100S ratios for these types and sizes of culverts. With this information, the designer of the road can select the most suitable and economical materials for bridging the stream.

Example 2

Situation: An engineer-designed, multipurpose forest road crosses a stream draining a 720-acre watershed. Maximum elevation is 5,450 ft.; stream gradient is 3 percent; elevation of the road surface on the centerline of the stream is to be 30.2 ft. above the stream channel at the culvert entrance; length of the culvert (if used) is 120 to 140 ft.; designed discharge is for a recurrence interval of 50 years. The channel below the crossing site contains no obstructions to flow.

Required: Discharge at the recurrence interval of 50 years, waterway area for a bridge, and culvert sizes and types.

From equation (9) or figure 5, the discharge at the recurrence interval of 50 years is 340 c.f.s. A waterway area of

$$\frac{340 \text{ c.f.s.}}{5 \text{ ft./sec.}} = 68 \text{ sq. ft.}$$

is required if the stream is to be spanned by a bridge. If the engineer chooses instead to use a culvert and does not want the depth of water to exceed the diameter of the culvert (i.e., a HW/D ratio of 1.0), a 96-inch culvert with projecting end or a 90-inch culvert with headwalls is required (tables 2 and 3). From table 5, a 9-ft. 6-inch by 6-ft. 5-inch pipe-arch of structural plate with projecting end or headwalls would carry the discharge. (The L/100S ratio is within the allowable limits for both the circular and the pipe-arch culverts.) If the engineer decides he will allow a headwater depth of twice the culvert diameter, a substantially smaller culvert can be used; a circular culvert of standard corrugated metal 72 inches in diameter with projecting end or headwalls would carry the designed discharge of 340 c.f.s. (tables 2 and 3). An 8-ft. 2-inch by 5-ft. 9-inch pipe-arch with projecting end or a 7-ft. by 5-ft. 1-inch pipe-arch with headwalls would also carry the designed discharge (table 5).

Example 3

Situation: Identical to example 2 except that the road surface will be 10 ft. above the channel surface of the culvert inlet, the HW/D ratio is 1.0, and fill depth over the culvert must be one-half the diameter of the culvert or one-half the span of the pipe-arch.

Required: Discharge at the recurrence interval of 50 years as previously determined, and culvert sizes.

In this situation, two or more culverts will be required and culvert diameter plus one-half culvert diameter must be less than 10 ft. If two circular culverts of corrugated metal are used, each must carry one-half of the designed discharge or 1.70 c.f.s. From tables 2 and 3, a circular culvert of corrugated metal 72 inches in diameter would carry 186 to 206 c.f.s. (depending on whether a projecting end or headwalls are used). The 6-ft. pipe diameter plus one-half pipe diameter equals 9 ft. Because this distance is less than the 10-ft. depth from stream channel to road surface, two 72-inch culverts could be used. Similarly, two 7-ft. by 5-ft. 1-inch pipe-arches could also be used (table 5).

SUMMARY

A method is presented for predicting flood discharge from the forested Blue Ridge Mountains of North Carolina for storms at recurrence intervals of 2.33, 5, 10, 20, 30, 40, and 50 years. These predictions are based on area and maximum elevation of the drainage. Once storm discharge has been estimated, the proper size of culvert can be determined from tables which list the discharge capacity of several types of culverts. Dimensions of the waterway area of bridges can also be estimated. Although these results were derived for the mountains of North Carolina, inclusion of discharge data from forested drainages of north Georgia, northwest South Carolina, and east Tennessee might be used to recalculate discharge relationships which would be applicable to a larger geographic area of the southern Appalachians.

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Accurate estimates of flood discharges from forested lands are needed for design of bridges and culverts. Equations incorporating watershed area and maximum elevation were developed for discharges at recurrence intervals of 2.33, 5, 10, 20, 30, 40, and 50 years from forested land in the Blue Ridge Province of North Carolina. These equations accounted for 98 percent of the variation in discharge. Capacity tables for several types and sizes of culverts are presented to simplify problems in culvert design.

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