FLOODS

AND

FLOOD CONTROL

Tennessee Valley Authority

TECHNICAL REPORT No. 26

KNOXVILLE, TENNESSEE · 1961

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Technical Reports Nos. 1, 2, and 3 were out of print at the time of this publication.

Arrangement and numbering of the TVA technical reports follows:

*Technical Reports Nos. 1-20—Dams**
Nos. 1-14 have been issued.
Nos. 15-20 are reserved for future dams.

Technical Reports Nos. 21-30—Manuals
Nos. 21-24 have been issued.
No. 25, *Navigation,* is scheduled for 1962.
Nos. 27-30 are reserved for future manuals.

*Technical Reports Nos. 31-40—Steam Plants**

*Except No. 8, *The Watts Bar Steam Plant.*

Printed in the United States of America

TENNESSEE VALLEY AUTHORITY,
Knoxville, Tenn., June 6, 1961.

MR. LOUIS J. VAN MOL, *General Manager,*
Tennessee Valley Authority, Knoxville, Tenn.

DEAR MR. VAN MOL: Technical Report No. 26, *Floods and Flood Control,* is one of a series of special reports prepared to cover certain phases of TVA's engineering and construction work in the unified development of the water resources of the Tennessee River system.

These special technical reports have been planned as a companion series to technical reports on the individual projects and record the results of experience gained on TVA projects in specialized fields over a period of years. It is recommended that Technical Report No. 26 be printed as a public document.

Yours very truly,

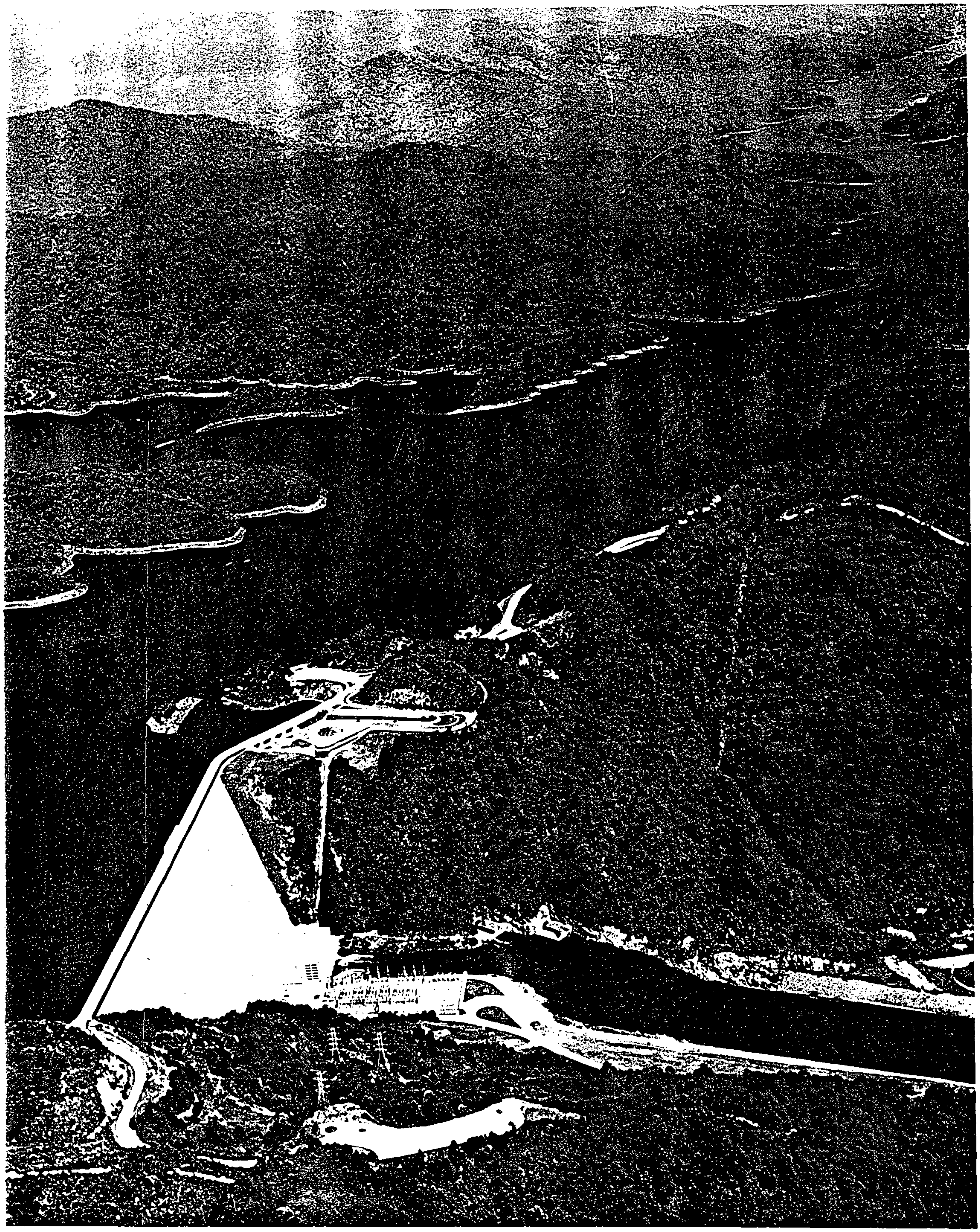
G. P. PALO,
Chief Engineer.

CONTENTS

APPENDIXES

ILLUSTRATIONS

TABLES



Fontana Dam—TVA's highest.

FLOODS AND FLOOD CONTROL
Tennessee Valley Authority

CHAPTER I

INTRODUCTION

Floods and Flood Control is one of a series of special technical reports covering important phases of the work of TVA's Office of Engineering. It was written by staff members of the Division of Water Control Planning and is the seventh volume of the series to be published.

An informative background to the report is provided by this introductory chapter which tells the primary purpose and principal concern of the report and then describes briefly important hydrologic and topographic characteristics of the Tennessee Basin, TVA's flood control system, and the over-all integrated water control system in the Basin. The scope of the report is presented by summaries of each chapter and appendix.

The primary purpose of the report's preparation is to present a discussion of the role of flood control in the multiple-purpose water control system in the Tennessee Valley for the use of experts in this field. The non-technical reader, however, may find useful information in much of the data contained herein, particularly in the data on past floods on the various streams.

The principal concern of the report is the technical engineering aspects of floods and flood control in the Tennessee River Basin and the effect of this control downstream on the lower Ohio and Mississippi Rivers. The report discusses at some length the great storms of the Valley and the historic floods that resulted from them. Included also are the methods that have been used in determining the maximum probable storms and consequent flood flows that are a determining factor in the design of dams and their appurtenances. The report is also concerned with the operating methods used and results obtained by TVA in regulating floods by use of the flood storage space provided in its system of multiple-purpose reservoirs. Discussions of related flood control aspects and factors appear at appropriate places throughout the report. The appendixes give statistics and more detailed information concerning TVA's actual flood control operations and important related procedures of the flood control program.

THE TENNESSEE RIVER BASIN

The Tennessee River system has its headwaters in the mountains of eastern Tennessee, western Virginia, western North Carolina, and northern Georgia. Two of the principal tributaries, the Holston and the French Broad Rivers, unite just above Knoxville, Tennessee, to form the Tennessee River. From Knoxville the Tennessee flows southwesterly through the State of Tennessee into northern Alabama where it turns to the northwest and flows into the northeast corner of Mississippi. There it swings north, again crosses Tennessee, and continues across Kentucky to Paducah where it enters the Ohio River (see fig. 1).

The entire drainage area covers 40,910 square miles and above Kentucky Dam—the farthest downstream of the system dams—it covers 40,200 square miles. It lies mostly in the State of Tennessee, although by no means does it cover that entire state—it also lies partly in the six other states mentioned.

Several headwater tributaries of the Tennessee originate high on the steep slopes of the Blue Ridge and Great Smoky Mountains, where some peaks rise to nearly 7,000 feet and where there is an abundant growth of hard and soft timber. Other headwater streams, notably those of the Clinch and Holston River systems, originate in the Great Valley of the Tennessee where long parallel wooded ridges alternate with valleys having scattered woodland and cultivated areas. The western half of the Basin is less rugged than the eastern portion, and substantial areas of flat or rolling land occur in middle Tennessee and along the Basin's western edge. Approximately 47 percent of the area west of Chattanooga is forested as compared with 56 percent east of that city.

Profiles of the Tennessee River and its principal tributaries (fig. 2) show a fall from the maximum reservoir surface at Thorpe Dam, highest upstream in the system, to the minimum tailwater surface at Kentucky Dam, lowest downstream in the system, of 3,192 feet in 714.2 river miles. The Tennessee River, commonly referred to herein as the main

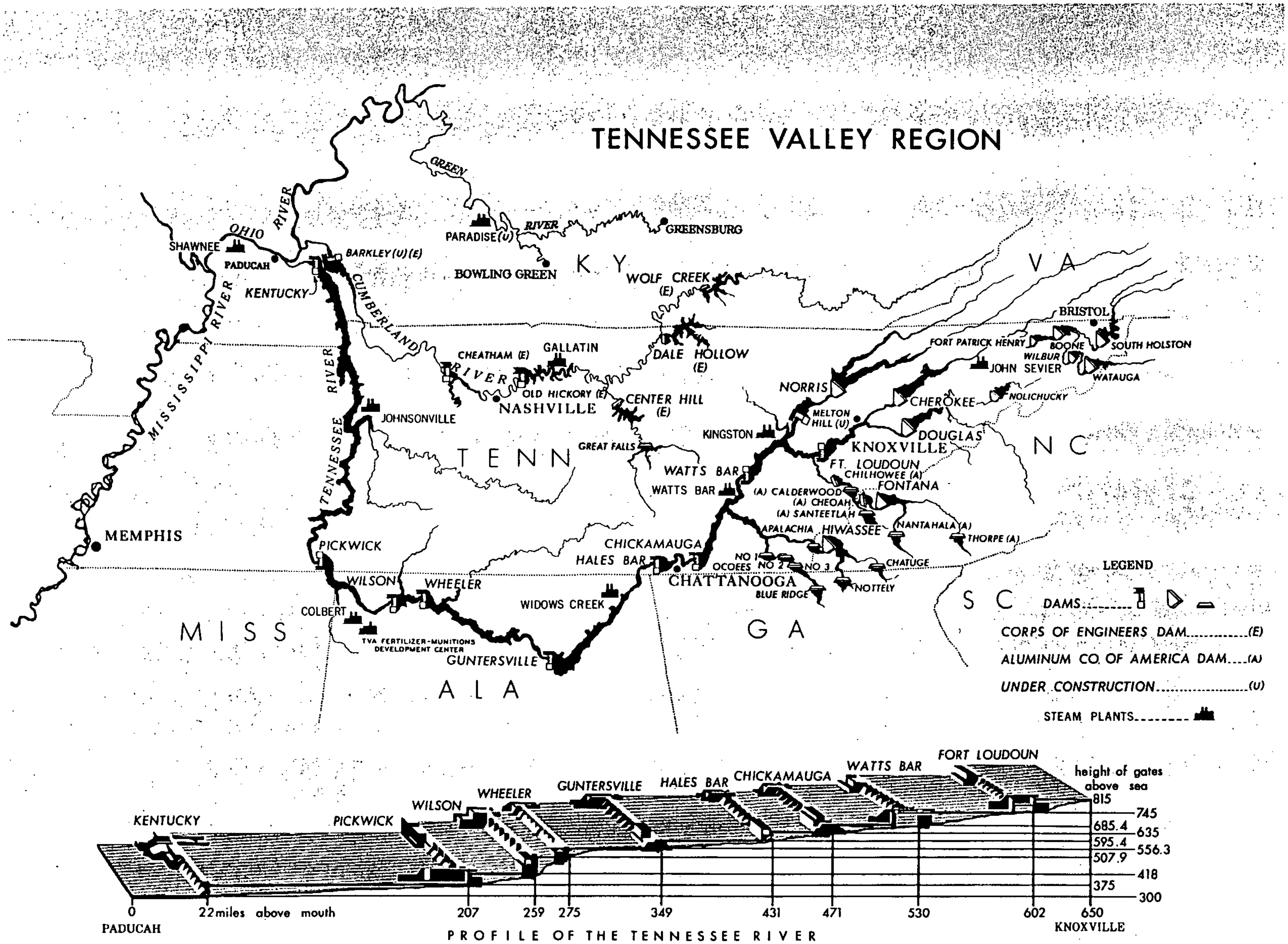


FIGURE 1.—*Major projects contributing to the TVA multiple-purpose system.*

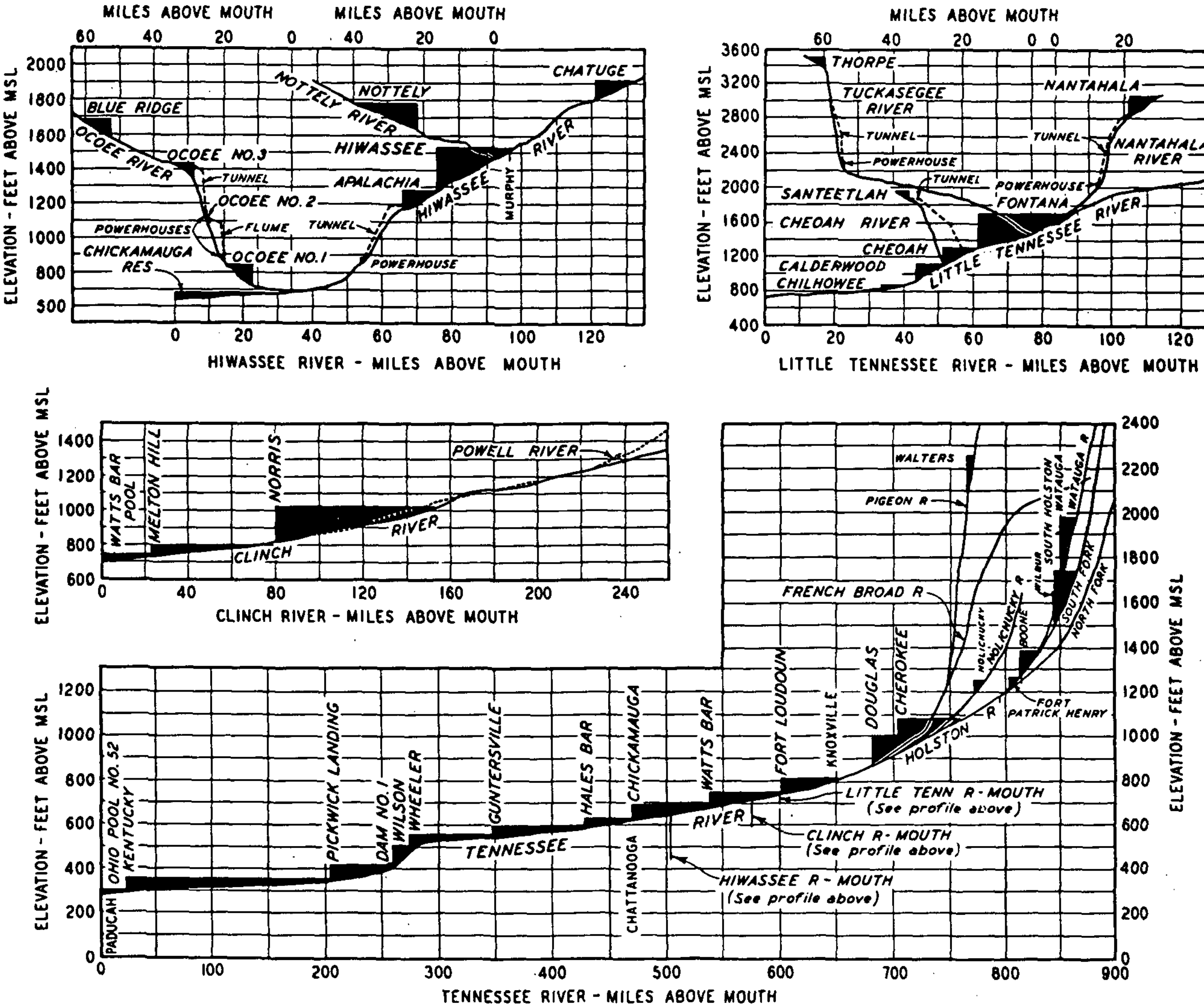


FIGURE 2.—*Profiles of the Tennessee River and its principal tributaries.*

river, has a fall from the top of the gates at Fort Loudoun Dam to the minimum tailwater at Kentucky Dam of 515 feet in 579.9 river miles.

Mean annual rainfall over the drainage area amounts to between 51 and 52 inches but has ranged from a low of 38 to a high of 65 inches. The heaviest precipitation occurs over limited mountainous areas along the headwaters of the tributaries where the mean annual reaches 80 to over 90 inches. In portions of the French Broad, Clinch, and Holston Valleys the mean annual is as low as 40 inches.

Mean annual runoff of the Tennessee River is about 42 percent of the precipitation over the drainage area. Although some snow falls in the mountainous areas, it does not accumulate to significant depths over large areas. Considerable natural storage is afforded, however, by the deep soils and other extensive underground storage in many of the tributary areas. This natural storage tends to stabilize the runoff to some extent. The dense ground cover on the steep slopes also helps to check rapid runoff from the heavy rainfall. Heavy storms moving across the Tennessee Valley between December and April become potential causes of widespread major floods in the Valley. Between June and October the area is subject to both cyclonic and local storms and to intense rains accompanying the passage of decadent hurricanes.

Natural river flow at the site of Kentucky Dam has ranged from a maximum of 500,000 cubic feet per second in 1897 to a minimum of 4,500 cubic feet per second in 1925, with an average of 65,500 from 1886 to 1959. At the Fort Loudoun dam site it varied from 1,600 cubic feet per second in 1925 to 300,000 cubic feet per second in 1867, with an average of 13,500 cubic feet per second from 1899 to 1959.

TVA'S FLOOD CONTROL SYSTEM

The TVA Act of May 18, 1933, specifically authorized the construction of ". . . dams, and reservoirs, in the Tennessee River and its tributaries . . ." that will best serve to ". . . control destructive flood waters in the Tennessee and Mississippi River drainage basins. . . ." In compliance with this authorization and directive, 18 multiple-purpose reservoirs in the over-all system have flood storage reservations. Eight of these reservoirs are on the Tennessee River and 10 are on tributaries and their total flood storage reservation is nearly 12,000,000 acre-feet on January 1 of each year. In addition, because of its beneficial regulating effect during most floods, flood control is included as one of the multiple purposes of another main river project, although it was not feasible to provide a flood storage reservation. Statistical data on the 19 projects in the flood control system are given in table 1, page 6, which lists the principal features of all the major projects in the integrated water control system in the Tennessee River Basin. The locations of the 19 projects in relation to the other projects in the integrated system and a view of each of the 19 are shown by figures 3 through 22 starting on page 8.

As to the system's flood control accomplishments, there are now 770 miles of river below the multiple-purpose dams where the agricultural lands on the bottoms on both sides of the stream have the frequency of flooding reduced greatly. The flood of February 1957 at Chattanooga would have reached a stage of 54 feet, the second highest of record under natural conditions, but was actually reduced nearly 22 feet to the point where only minor damage occured. Substantial but smaller reductions have been achieved at Chattanooga in 32 other floods during the 25 years from 1936 to 1961. The stage during floods greater than any that have been experienced can be reduced as much as 20 feet with the reservoirs now in existence upstream. At Cairo the great flood of January 1950 was reduced 1.9 feet. The flood of May 1958 was reduced 3.1 feet and, although it was a smaller flood than that of 1950, it was more significant in prevented damages because it occurred later in the year. In other floods the stage at Cairo can be reduced up to 4 feet, depending on the contribution of the Tennessee River to the peak on the Ohio River. A discussion of the monetary value of damages averted by flood control is included in the report.

Statements have been made that the Tennessee Valley is now completely protected from floods. A moment's consideration will show the fallacy of such claims. During floods the TVA reservoirs have generally reduced stages on the streams below the dams several feet, the amount of reduction depending on the magnitude and location of the storm rainfall causing the flood. These reductions have made supplemental protection unnecessary in some instances, while in others they have made it economically possible to provide such needed protection in the future. However, except in those areas immediately downstream from some of the large flood storage reservoirs, complete protection from floods is not assured.

On streams in the Basin where no control by reservoirs has been provided and upstream from the uppermost storage reservoirs, flood stages of course will—for the same flood flows—rise to the same heights as previously, in fact, the present-day stages may be higher than in the past because of obstructions that various interests have built in the channel and flood plain during recent years.

Except for Kentucky Reservoir, the main river has no storage reservoirs comparable in capacity per square mile of drainage area with those on the tributaries. Consequently, as a major flood proceeds down the main river and is joined by the flow from hundreds of smaller streams on which there is no control, the effect of upstream storage on the flood becomes less and less. For example, in a flood such as that of February 1957, the reduction at Chattanooga was nearly 22 feet, but at Florence—208 miles farther downstream—it was less than 5 feet.

Generally speaking, to have provided a greater degree of protection than that afforded by the system of storage reservoirs as built on the main river would not have been economical. The cost of land and relocation of communities, railroads, highways, and other features would have been too great. This is also the reason for the use of levees as the principal method of protection on the lower Mississippi. The Kentucky project, however, affords unusually large benefits to lands downstream along the lower Ohio and Mississippi Rivers because of its unique location near these two rivers and its large low-cost flood storage capacity. Kentucky is thus capable of regulating the outflow of the Tennessee into the two larger streams with assurance of dependable flood crest reduction of sufficient magnitude to justify the flood control portion of the cost of the project.

Even with the reductions in stage on the Tennessee River, as cited in the preceding paragraphs, there yet can be serious damage in communities along its banks during extreme floods.

INTEGRATED WATER CONTROL SYSTEM — TENNESSEE RIVER BASIN

The integrated water control system in the Tennessee River Basin consists of 26 major dams and reservoirs built or acquired by TVA, 9 on the main stem of the Tennessee River and 17 on tributaries—construction of one of these latter is still underway with completion scheduled for 1963. In addition, 6 major projects of the Aluminum Company of America in the tributary Little Tennessee River Basin are operated under TVA instructions by agreement. Of the total 32 projects, 19 are those previously mentioned as comprising the flood control system. All 32 projects are shown in figure 1 and their principal features are given in table 1. Other major projects which contribute power to the system are also shown in figure 1. There are also 13 minor dams in the Basin which contribute power to the system.

The over-all planning with respect to both design and operation of the flood control and other system projects built by TVA was—and is—a primary function of TVA's Division of Water Control Planning. This division makes engineering investigations and surveys—including the collection of basic technical engineering data. These provide the bases from which the division plans and recommends the location, size, feasibility and appurtenances of projects for TVA's river control and power production facilities. This division is also responsible for the scheduling and issuing of instructions concerning reservoir operations.

SCOPE OF THE REPORT

The following summaries of the chapter and appendix contents present the scope of *Floods and Flood Control.*

Chapter 1, "Introduction," provides background information for the report. It gives its primary purpose and principal concern; briefly describes the Tennessee River Basin, TVA's flood control system, and the over-all integrated water control system in the Basin; and presents the scope of the report in some detail.

Chapter 2, "Flood - Producing Storms," is a comprehensive discussion of the storms which cause Tennessee Valley floods. The chapter presents in some detail the principal characteristics of these storms, it summarizes Valley-wide rainfall data, describes outstanding storms which produced floods prominent in the Valley's flood history, and briefly discusses intense storms over small areas. The last section of the chapter describes how observed storm data can be used to determine probable future extreme storms for a watershed. Such storms would produce floods of a magnitude for which estimated data would be suited for the design of water control and flood protection projects.

Chapter 3, "Sources of Flood Data," begins with a summary of the uses for which flood experience data are required—particularly as they apply in the Tennessee River Basin. It then discusses Valley flood history investigations and sources of both historical and recorded flood data. The chapter ends with a brief description of the preparation of flood profiles which the following chapter discusses in detail.

Chapter 4, "Tennessee Basin Floods," divides these floods into two categories—main river and tributary. With respect to the main river, the chapter tells about the preparation of flood profiles and discusses flood volumes and frequencies. The tributary flood section of the chapter describes each major tributary basin, outlines its flood history, and presents tabulated data on maximum known floods that occurred therein.

Chapter 5, "Design Flood Flows," covers the determination of these flows which are basic in the development of any plan for flood protection. The chapter first discusses the characteristics affecting flood runoff in the Valley. It then describes the methods TVA used to determine maximum flood flows for design purposes. This latter section defines the three flood designations—design, maximum probable, and maximum possible—considered in planning TVA projects.

Chapter 6, "Computation of Natural Flood Hydrographs," outlines the methods used by TVA in computing progressive variations in flow for natural floods and discusses flood routing. These discussions include fundamental routing procedures and—for the Tennessee River—the computation of natural storage, inflow for pre-reservoir floods, and discharge.

Chapter 7, "Studies and Principles of Flood Control Operations," first summarizes briefly the flood situation in the Valley and on the lower Ohio and Mississippi Rivers before TVA. It then itemizes the reservoir operating principles and objectives which emerged as planning progressed and operating experience developed, and discusses the principal factors considered and studied in their determination. These factors include the effect of section 7 of the Flood Control Act of December 22, 1944. The remainder of the chapter discusses fixed-rule and ideal reservoir operation, and the coordination of flood control with operations for navigation, power, malaria control, and recreation.

Chapter 8, "Flood Control Storage and Its Use," is devoted to the capacity of the system with respect to flood control. Following a brief summary of the projects in the system, the discussions relate to storage capacity required and its distribution above and below Chattanooga, including that proposed for Asheville and other areas along the upper French Broad River; outlet works capacity; and the functions of the two groups of reservoirs—main river and tributary.

Chapter 9, "Actual Reservoir Operation for Flood Control," starts with discussions covering the application of operating principles using the general methods described in Chapter 7. The next section of the chapter describes the many factors entering into the mechanics of TVA reservoir operation, and includes an itemization of the work involved. The concluding discussions concern the problems — both temporary and continuing—affecting flood control operations.

Chapter 10, "Local Flood Problems in the Tennessee River Basin," first outlines the principal objectives of flood control and discusses several alternative methods of preventing flood damage. Then there is a brief summary of TVA's preparation of reports to aid Valley communities solve flood problems not

TABLE 1.—*Principal features of major projects—integrated water control system—Tennessee River Basin (names of flood control projects are in capital letters).*

DAM AND APPURTENANCES

PROJECT	DATE OF FIRST USE [a]	RIVER	MAX. [b] HEIGHT (FEET)	OVERALL CREST LENGTH (FEET)	MAX. [c] SPILLWAY CAPACITY (CFS)	VOLUME OF CONCRETE (CU. YDS.)	VOLUME [d] EARTH AND/OR ROCK FILL (CU. YDS.)	POWER [e] INSTALLED CAPACITY (KW)	POWER [e] ULTIMATE CAPACITY (KW)	LOCK SIZE (FEET)	LOCK MAX. LIFT (FEET)
KENTUCKY	1944	Tennessee	206	8422	1,050,000	1,356,000	5,582,000	160,000	160,000	110x600	75
PICKWICK	1938	Tennessee	113	7715	650,000	679,100	3,081,000	216,000	216,000	110x600	63
WILSON	1925	Tennessee	137	4535	686,000[pp]	1,280,000	0	436,000	598,000[kk]	110x600 60x300 / 60x292	100 100[k]
WHEELER	1936	Tennessee	72	6342	542,000	808,400	0	259,200	356,400[mm]	60x400 / 110x600	52 / 52 [rr]
GUNTERSVILLE	1939	Tennessee	94	3979	478,000	308,600	874,900	97,200	97,200	60x360	45
HALES BAR	1913	Tennessee	112	2315	224,000	j	j	99,700	99,700	60x265	41
CHICKAMAUGA	1940	Tennessee	129	5800	470,000	506,400	2,793,500	108,000	108,000	60x360	53
WATTS BAR	1942	Tennessee	112	2960	560,000	480,200	1,210,000	150,000	150,000	60x360	70
FORT LOUDOUN	1943	Tennessee	122	4190	390,000	586,700	3,594,000	128,000	128,000	60x360	80
Apalachia	1943	Hiwassee	150	1308	136,000	237,800[p]	0	75,000	75,000		
HIWASSEE	1940	Hiwassee	307	1376	112,000[t]	800,600	0	117,100[jj]	117,100[jj]		
CHATUGE [u]	1942[v]	Hiwassee	144	2850[w]	11,500	25,700	2,347,400	10,000	10,000		
Ocoee No. 1	1912	Ocoee	135	840	45,000[y]	160,000	0	18,000	18,000		
Ocoee No. 2	1913	Ocoee	30	450	j	0	0	21,000	21,000		
Ocoee No. 3	1943	Ocoee	110	612	95,000	82,500[p]	82,000	27,000	27,000		
Blue Ridge	1931	Toccoa	167	1000	55,000	j	1,500,000	20,000	20,000		
NOTTELY [u]	1942[v]	Nottely	184	2300[bb]	11,500	21,700	1,552,300	15,000	15,000		
Melton Hill [qq]	1963	Clinch	80	1072	124,000	250,000	0	72,000	72,000	75x400	60
NORRIS	1936	Clinch	265	1860	93,400[t]	1,002,300	181,700	100,800	100,800		
Chilhowee [cc]	1957	Little Tenn	91	1373	182,000	91,500	307,000	50,000	50,000		
Calderwood [cc]	1930	Little Tenn	232	916	260,000	j	0	121,500	121,500		
Cheoah [cc]	1919	Little Tenn	225	750	200,000	j	0	110,000	110,000		
FONTANA	1945	Little Tenn	480	2365	134,300[t]	2,815,500	760,600	202,500	202,500		
Santeetlah [cc]	1928	Cheoah	212	1054	76,100	j	0	45,000	45,000		
Nantahala [cc]	1942	Nantahala	250	1042	59,000	j	1,829,000	43,200	43,200		
Thorpe [cc]	1941	Tuckasegee	150	900	56,000	j	1,060,000	21,600	21,600		
DOUGLAS	1943	French Broad	202	1705[ee]	342,000[t]	556,400[p]	127,900[p]	112,000	112,000		
Nolichucky	1913	Nolichucky	j	j	j	j	j	10,640	10,640		
CHEROKEE	1942	Holston	175	6760[gg]	286,000[t]	694,200	3,304,100	120,000	120,000		
Fort Patrick Henry	1953	S Fork Holston	95	737	141,000	72,500	30,400	36,000	36,000		
BOONE	1953	S Fork Holston	160	1532	137,000	198,400	714,000	75,000	75,000		
SOUTH HOLSTON	1951	S Fork Holston	285	1600	116,000[t,hh]	97,500	5,897,400[p]	35,000	35,000		
Wilbur	1912	Watauga	77	375	34,000	j	j	10,700	10,700		
WATAUGA	1949	Watauga	318	900	73,600[t,hh]	80,400	3,497,800	50,000	50,000		

RESERVOIR DATA AND OPERATING LEVELS

AREA AT TOP OF GATES (ACRES)	TOTAL VOLUME BELOW TOP OF GATES (ACRE-FEET)	USEFUL [f] CONTROLLED STORAGE (ACRE-FEET)	LENGTH [g] OF SHORE LINE (MILES)	BACKWATER LENGTH (MILES)	MAXIMUM CONTROLLED POOL LEVEL (EL.)	MINIMUM [h] EXPECTED POOL LEVEL (EL.)	AVERAGE TAILWATER LEVEL (EL.)	HEAD [i] (FEET)	PROJECT
261,000	6,002,600	4,010,800	2,380	184.3	375	354	310	47	KENTUCKY
46,800	1,091,400	418,400	496	52.7	418	408	362	50	PICKWICK
16,000	650,000	53,000	154	15.5	507.88	504.5	414	92	WILSON
68,300	1,150,400	347,500	1,063	74.1	556.28	550	507	48	WHEELER
70,700	1,018,700	162,900	962	82.1	595.44	593	557	37	GUNTERSVILLE
6,680	147,660[m]	12,370[m]	162	39.9	635[n]	632	596	35	HALES BAR
39,400	705,300	329,400	810	58.9	685.44	675	634	45	CHICKAMAUGA
43,100	1,132,000	377,600	783	72.4	745	735	682	56	WATTS BAR
15,500	386,500	109,300	360	55.0	815	807	740	70	FORT LOUDOUN
1,123	58,700	8,700	31	9.8	1280	1272	840[r]	380[s]	Apalachia
6,280	438,000	364,700	180	22	1526.5	1415	1275	254	HIWASSEE
7,150	247,800	229,300	132	13	1928	1860	1804	126	CHATUGE
1,900	91,300	33,100	18	7.5	837.65	816.9	724	113	Ocoee No. 1
j	j	0[z]	j	0	1115	j	843[r]	252	Ocoee No. 2
604	8,700[aa]	5,850[aa]	24	7	1435	1413	1119[r]	313	Ocoee No. 3
3,320	200,800	186,300	60	10	1691	1590	1543	147	Blue Ridge
4,290	184,400	171,300	106	20	1780	1690	1612	174.5	NOTTELY
5,720[m]	118,600[m]	26,000[m]	144	44	796[n]	790	742	51	Melton Hill [qq]
40,200	2,567,000	2,281,000	800	72 Clinch 56 Powell	1034	930	826	194	NORRIS
1,690	49,250	6,564	30	8.9	874	870	812	60[s]	Chilhowee [cc]
536	41,160	1,570	j	8	1087.5	1084.5	869[r]	209[s]	Calderwood [cc]
595	35,030	1,850	j	10	1276.5	1273.5	1087	187[s]	Cheoah [cc]
10,670	1,444,300	1,157,300	248	29	1710	1525	1276	429	FONTANA
2,863	158,250	133,300	85	7.5	1939.92	1863.0	1275[r]	597[s]	Santeetlah [cc]
1,605	138,730	126,000	j	4.6	3012.16	2881.0	2007[r]	944[s]	Nantahala [cc]
1,462	70,810	67,100	j	4.5	3491.75	3415.0	dd	1200	Thorpe [cc]
31,600	1,514,100	1,419,700	555	43.1	1002	920	873	129	DOUGLAS
797	9,850	j	j	j	1245.9	j	j	68	Nolichucky
31,100	1,565,400	1,473,100	463	59	1075	980	925	149	CHEROKEE
893	27,100	4,300	36.9	10.3	1263	1258	1195	75	Fort Patrick Henry
4,520	196,700	150,000	130	17.3 S Fork Holston 15.3 Watauga	1385	1330	1264	123	BOONE
8,750	744,000	625,200	168	24.3	1742	1616	1490	239	SOUTH HOLSTON
72	j	327	3	1.75	1650	1645	1585	62	Wilbur
7,200	678,800	627,200	106	16.7	1975	1815	1650	309	WATAUGA

a. First unit placed in commercial operation.
b. From deepest excavation on or near base line to roadway or deck.
c. At maximum controlled pool level.
d. Includes riprap.
e. Based on rated capacity.
f. From maximum controlled level to minimum expected pool level.
g. At clearing line elevation.
h. Except during drawdown in advance of floods at main-river plants.
i. Head at maximum power storage level of tributary storage projects and average head at tributary *run-of river and main-river projects.*
j. No definite figure available.
k. Two-lift auxiliary lock.
m. At maximum allowable pool level.
n. Applies to top of gates. Maximum allowable pool elevation is 1 foot lower.
p. Apalachia - An additional 210,600 cubic yards concrete in tunnel.
Ocoee No. 3 - An additional 28,500 cubic yards concrete in tunnel.
Douglas - An additional 3,800 cubic yards concrete in saddle dams and dike.
An additional 797,200 cubic yards fill in saddle dams and dike.
South Holston - An additional 205,300 cubic yards fill in saddle dam.
r. At remote powerhouse.
s. *Net head.*
t. Includes capacity of discharge conduits.
u. Storage projects initially.
v. Closure date.
w. Saddle dams and spillway, 1480 feet additional.
y. Ocoee No. 1 - At high water elevation 834.6.
z. Reservoir silted.
aa. 1953 data.
bb. Saddle dam, 340 feet additional.
cc. Property of Aluminum Company of America; operation coordinated with TVA system.
dd. Center line of nozzle elevation 2284.4.
ee. Saddle dams, 3670 feet additional.
gg. Saddle dams, 1770 feet additional.
hh. Surcharged pool for maximum design flood.
jj. Unit 2 is a reversible pump-turbine.
kk. Includes 3 new units of 54,000-kw capacity each, on which work started March 9, 1959.
mm. Includes 3 new units of 32,400-kw capacity each, on which work started February 25, 1960.
pp. 50 gates (reduced from 58 because of new lock).
qq. Construction started September 6, 1960.
rr. Land wall of existing small lock failed June 2, 1961--rebuilding now underway. Construction of new large lock started October 3, 1960.

JULY 1961

taken care of by TVA's reservoir system. The last section gives brief descriptions of flood problems and possible methods of reducing damage at many Tennessee River Basin communities subject to damaging floods.

Chapter 11, "Effect of Changes in Land Use on Floods," devotes the larger part of its discussions to TVA's land use improvement efforts, particularly to its agricultural and forestry improvement programs as they relate to the effect on floods of land use changes. Preceding these discussions, however, the chapter briefly discusses flood control aspects of land use, extreme floods and land use, the relation of erosion and land use to flood control, and the effect of improved land use on floods. The latter part of the chapter appraises land use effect on large areas using data from small watersheds, and discusses the present method for determining the influence on flood control of improved land use.

Chapter 12, "Damage From Floods," briefly traces the growth of flood damage as flood plain development intensifies; it classifies flood damages and outlines the bases of TVA's damage estimates; it discusses potential flood damages in Chattanooga in relation to periodic appraisals for that city; it explains how annual preventable flood damages at critical locations are determined, and summarizes these damages; and concludes with a brief discussion of future flood damage.

Chapter 13, "Benefits From Flood Control," first relates damages to benefits and classifies benefits, it then presents the many factors affecting benefit calculation. The remainder of the chapter discusses actual benefits—resulting from TVA's flood control system—in the Tennessee Valley and below Kentucky Dam on the Tennessee and lower Ohio and Mississippi Rivers; and incidental benefits to other water use programs. A summary of the average annual flood control benefits ends the chapter.

Chapter 14, "Cost and Economics of Flood Control," covers many ramifications of this subject. Its discussions include average annual costs, TVA's method of allocating multiple-purpose project costs, allocation of investment to flood control, annual charges for flood control, benefit-cost ratio, and return on investment. Summaries of flood control accomplishments in relation to benefits and costs, and of flood control operations as they affect water power and land, conclude this final chapter of the report.

Appendix A, "Typical Flood Control Operation January 1947 and Results of Flood Regulation January 1946," is primarily the day by day story of regulation of the Tennessee Basin flood caused by the storm of January 14 to 20, 1947. This regulation is considered typical of TVA's flood control operations. The appendix also gives a brief summary of the successful regulation of the even higher January 1946 flood.

Appendix B, "Possible Flood Crest Reduction," presents numerical and graphical data concerning Tennessee Valley pre-reservoir and post-reservoir floods, these data resulting principally from the application of methods and principles described or outlined in the body of the report. Among these data are hydrographs showing operation of the system reservoirs during the 1950 flood—one which tested the system. The discussions cover methods of computing flood reductions, and results of actual operation—including a comparison with planning studies—of the reservoirs for flood control. A brief discussion of the operation of proposed detention basins above Asheville on the French Broad River concludes the appendix.

Appendix C, "Flood Damage Appraisal, Chattanooga, Tennessee, 1938," includes that part of the published report, "The Chattanooga Flood Control Problem" (House Document No. 91, 76th Congress, 1st Session, 1939), which describes the methods used in making the 1938 flood damage appraisal of Chattanooga. This appraisal is the basis for the determination—as discussed in chapter 12—of potential flood damage in that city. The methods used in appraising various types of flood damage are first discussed and the appraisal data then applied to past floods and to the design flood. The concluding discussion covers intangible flood damage.

Appendix D, "Report on Allocation of Costs as of June 30, 1953, Pursuant to Section 14 of the Tennessee Valley Authority Act, and Notes on That Allocation," is TVA's most recent—as of June 30, 1953—allocation report which was prepared upon completion of Boone, the latest multiple-purpose project (to 1961) to be placed in operation.

FLOOD CONTROL PROJECTS IN THE TENNESSEE RIVER SYSTEM

This section includes:

Table 1—listing all major projects in the integrated water control system in the Tennessee River Basin, giving the principal features of each. The 19 flood control projects are shown in capital letters.

Figure 3—showing diagrammatically the projects listed in table 1 with the 19 flood control projects identified by bold type, and the 18 reservoirs with flood control reservations shown by darker shading.

Figures 4 through 22—a view of each flood control project.

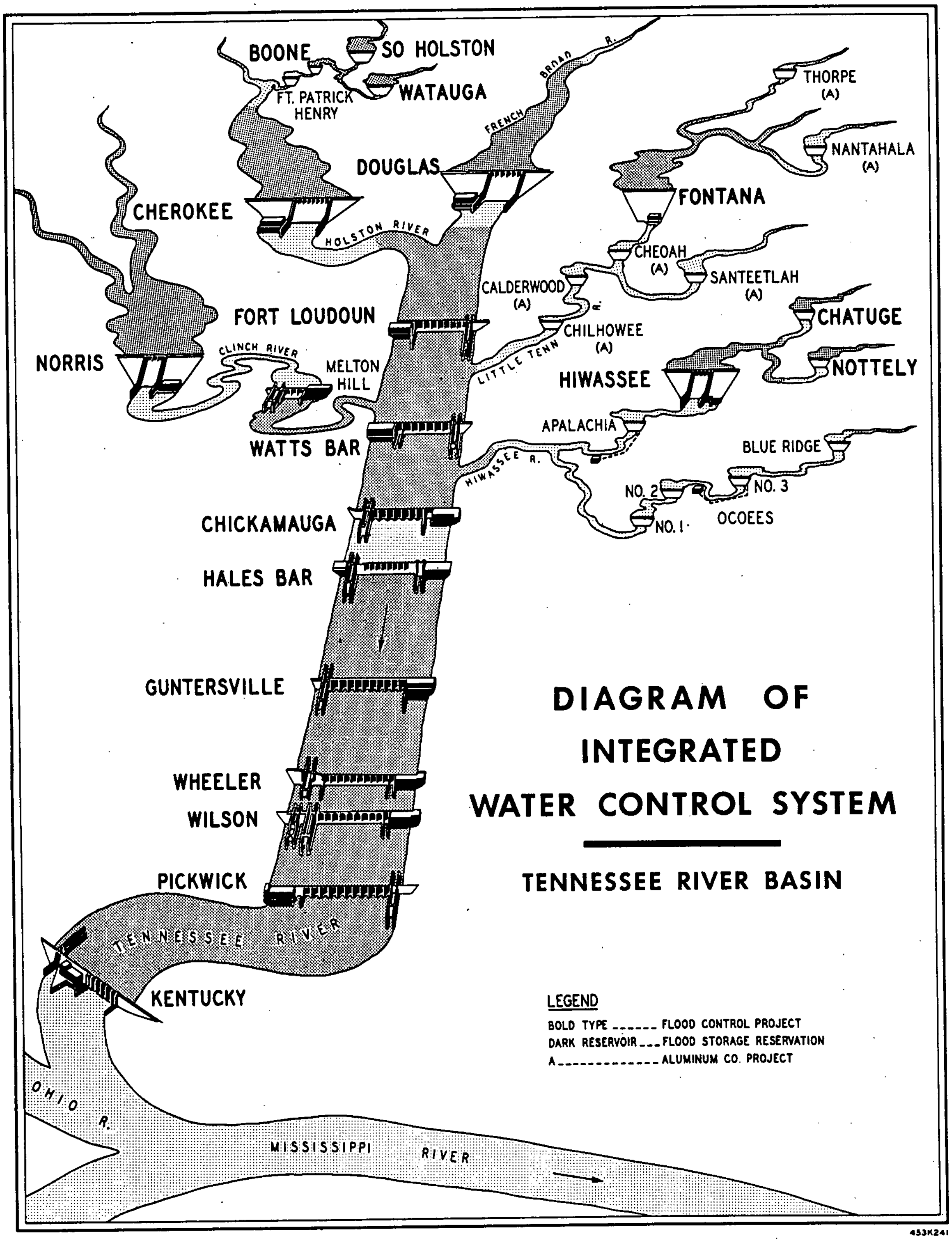


FIGURE 3.—*Diagram showing location of flood control projects in the Tennessee Basin integrated water control system.*

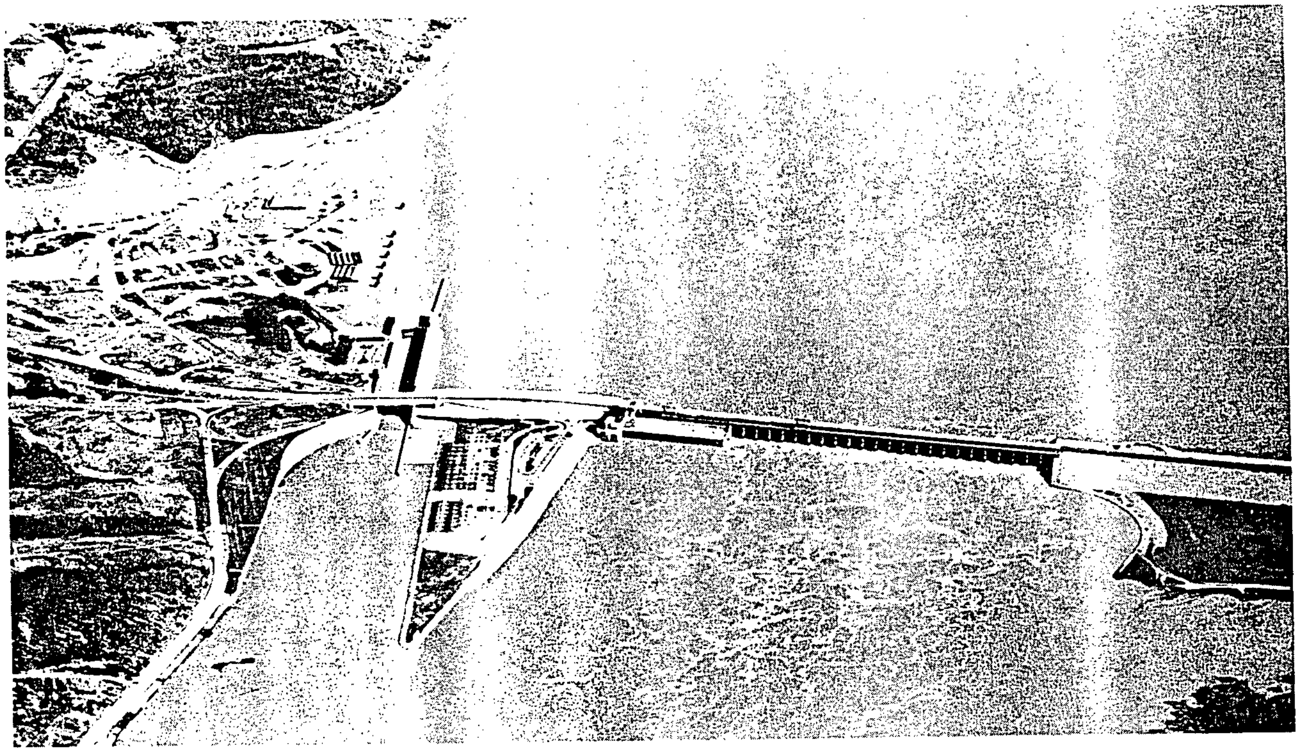


FIGURE 4.—*Kentucky—TVA's largest.*

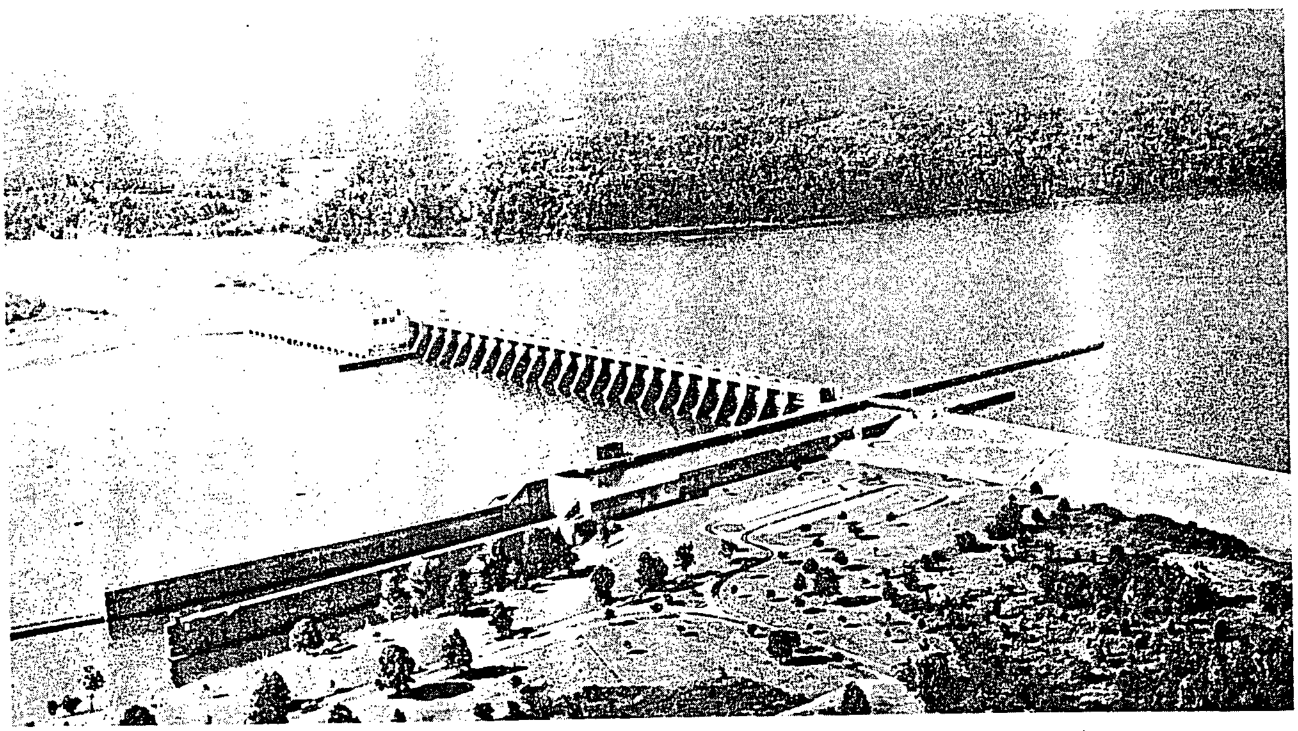


FIGURE 5.—*Pickwick Landing.*

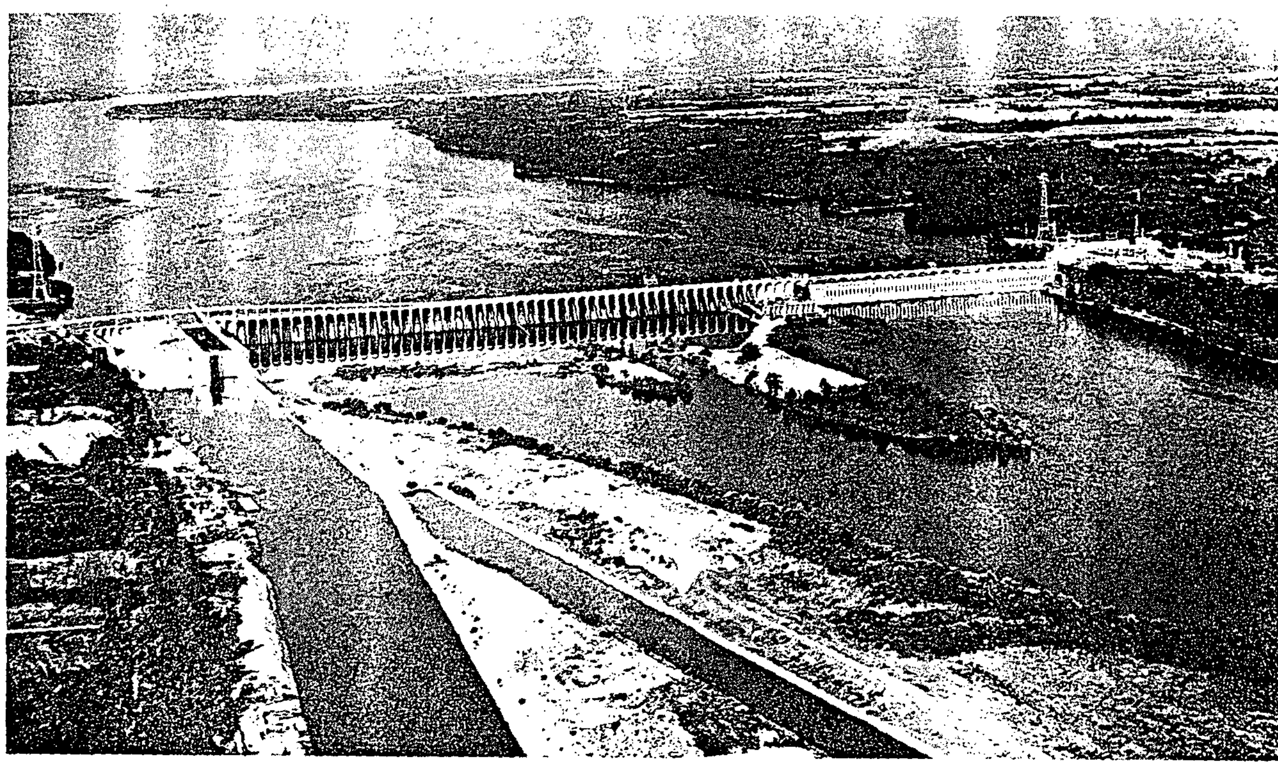


FIGURE 6.—*Wilson.*

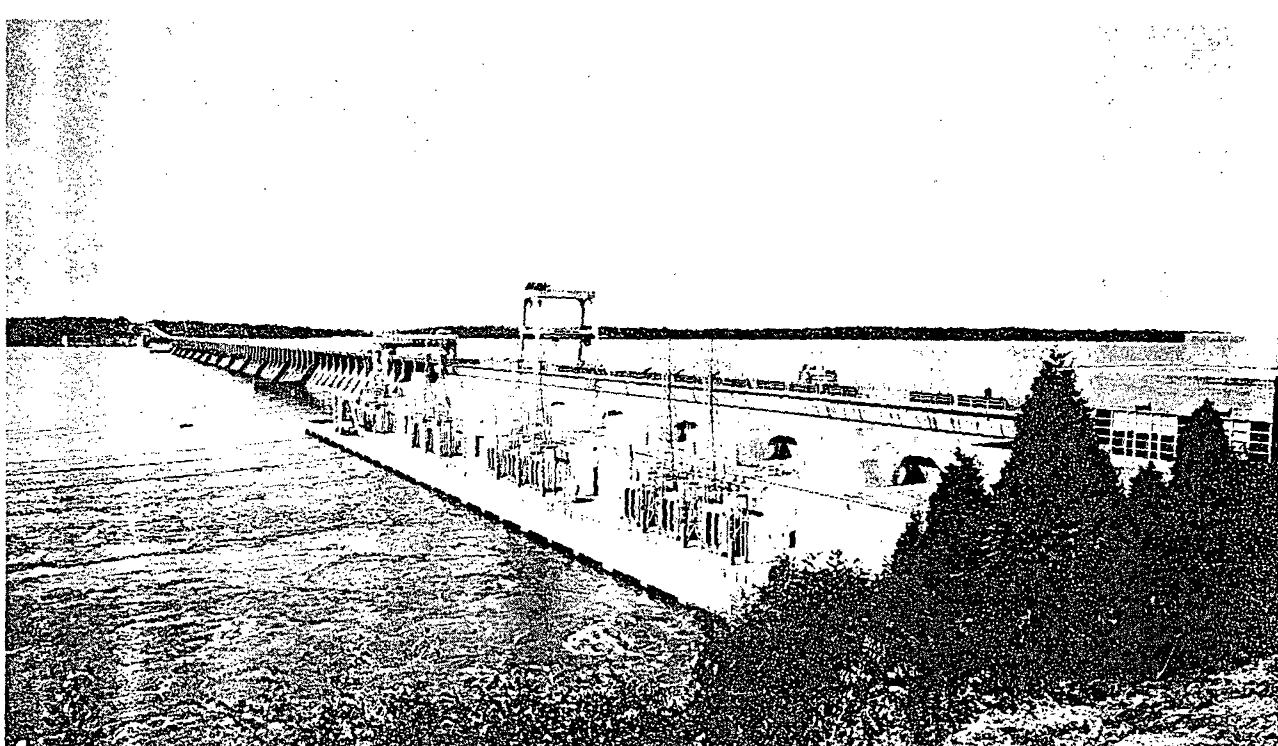


FIGURE 7.—*Wheeler.*

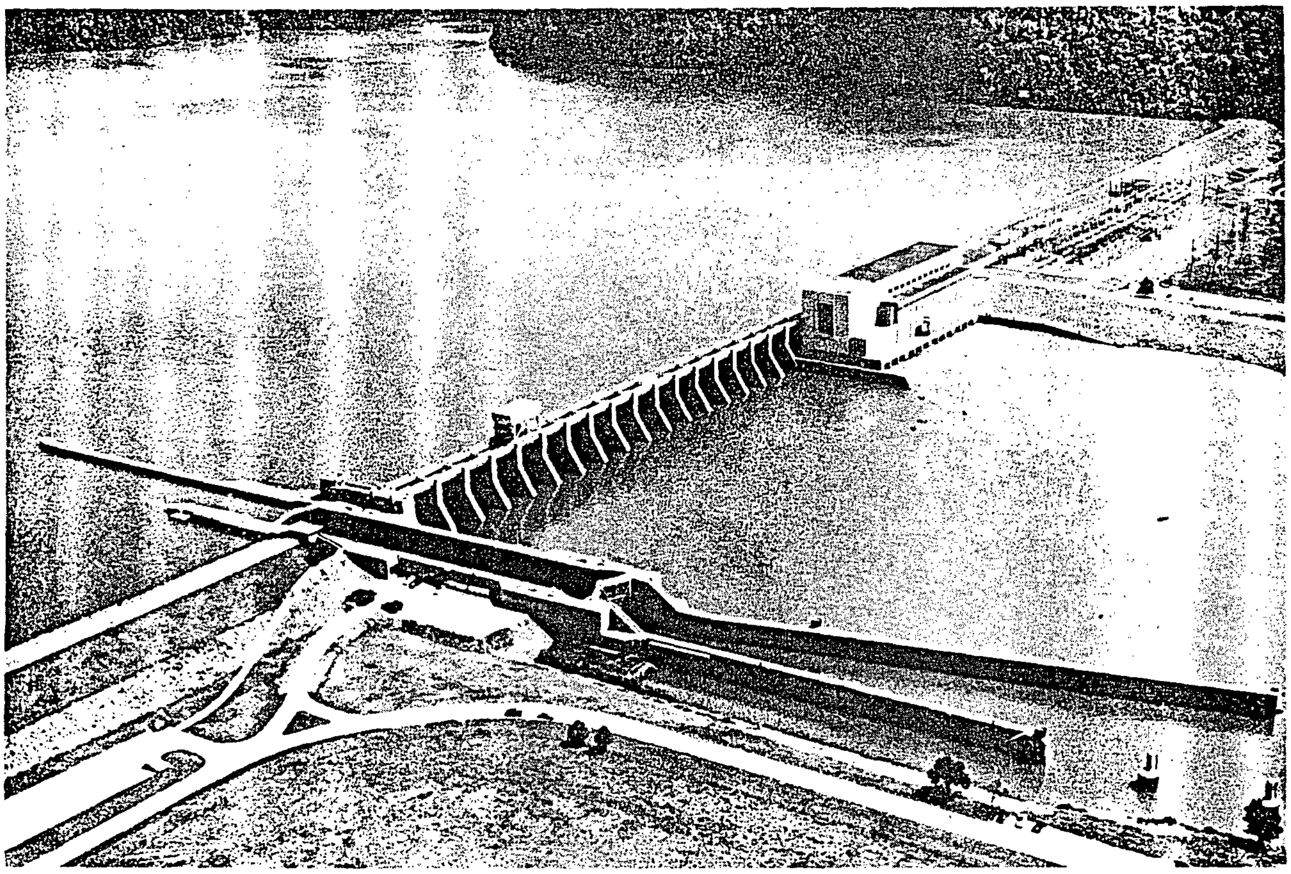


FIGURE 8.—*Guntersville.*

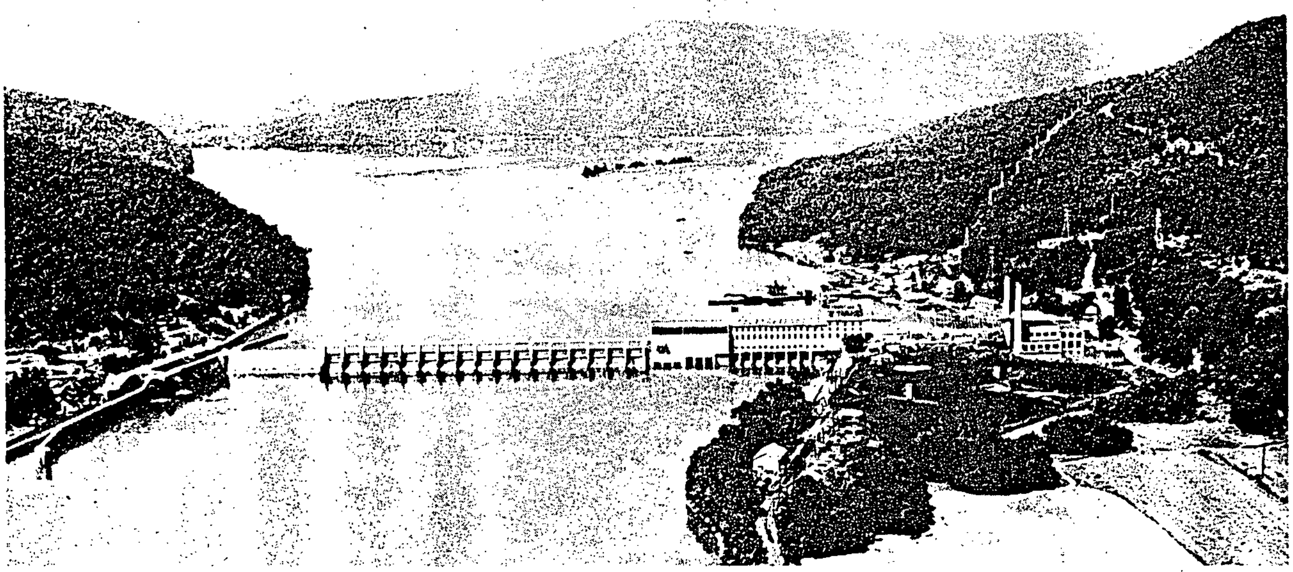


FIGURE 9.—*Hales Bar.*

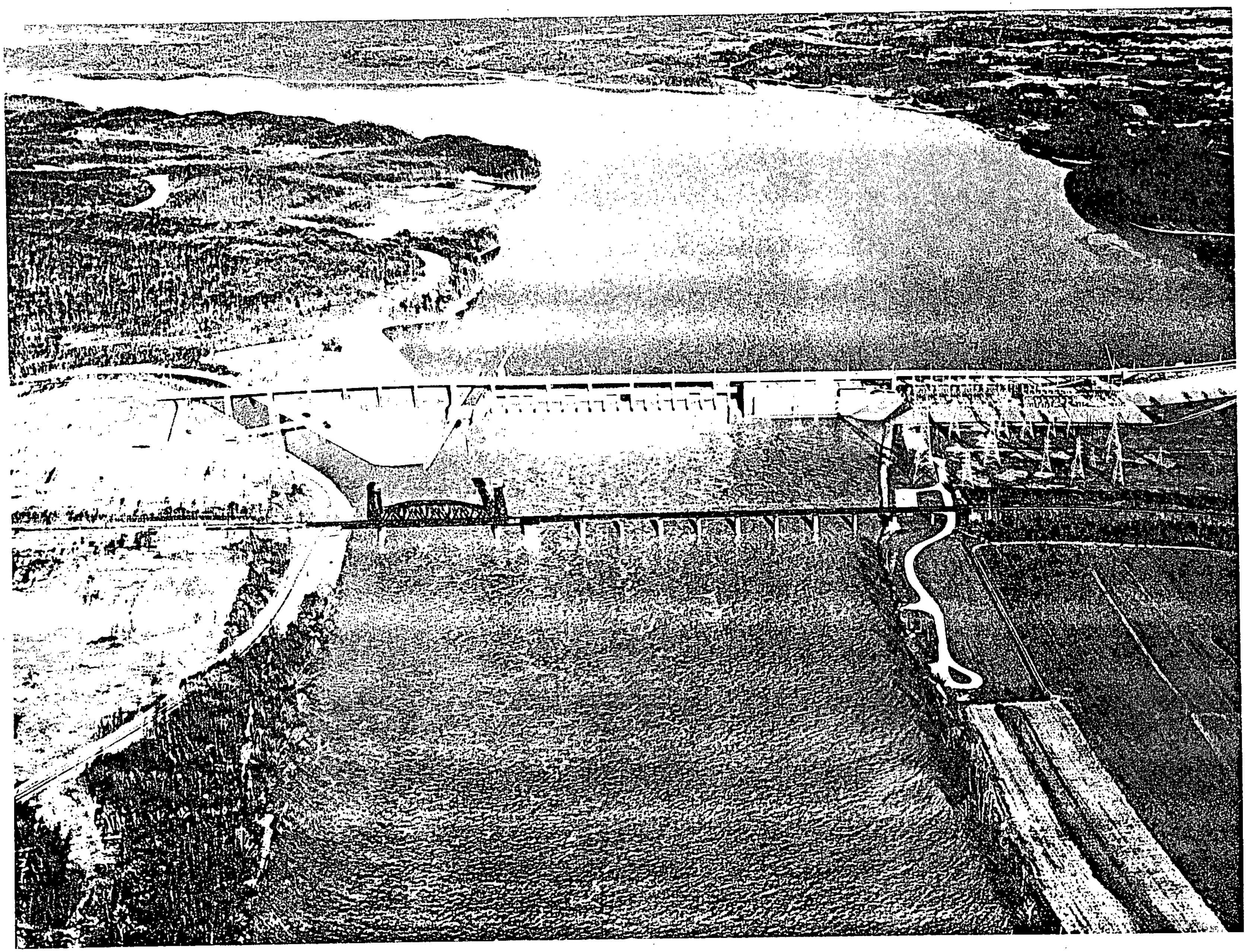


FIGURE 10.—*Chickamauga.*

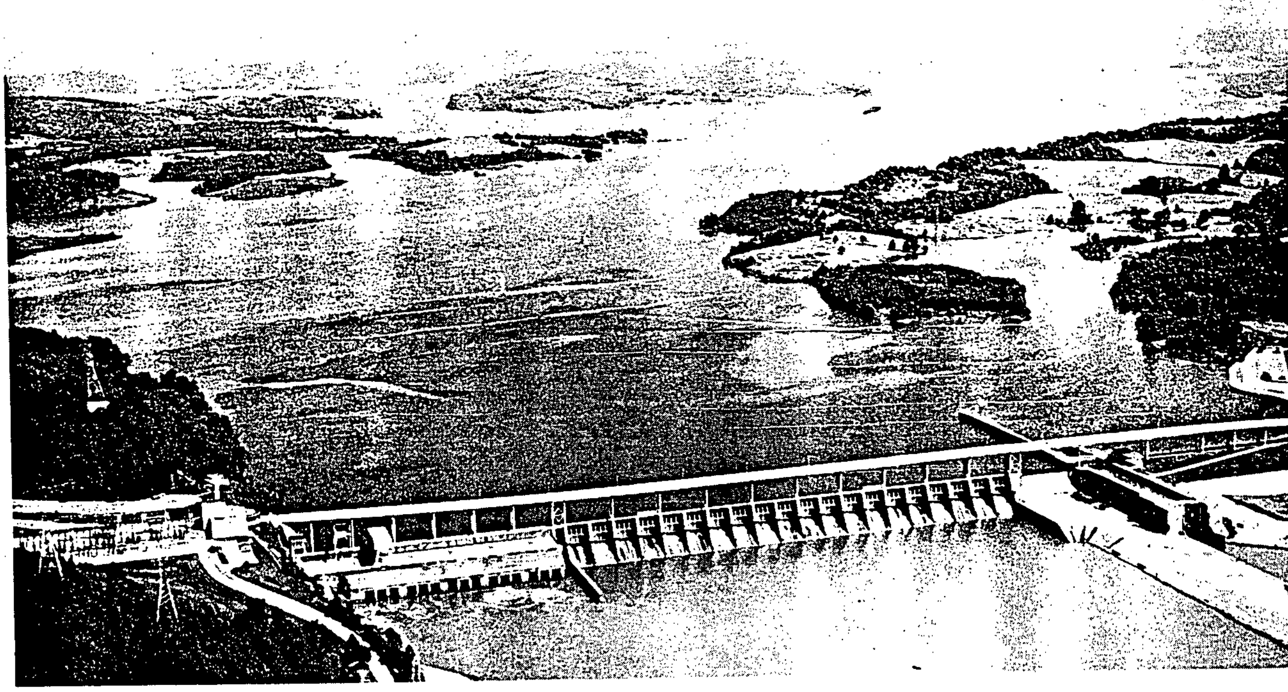


FIGURE 11.—*Watts Bar.*

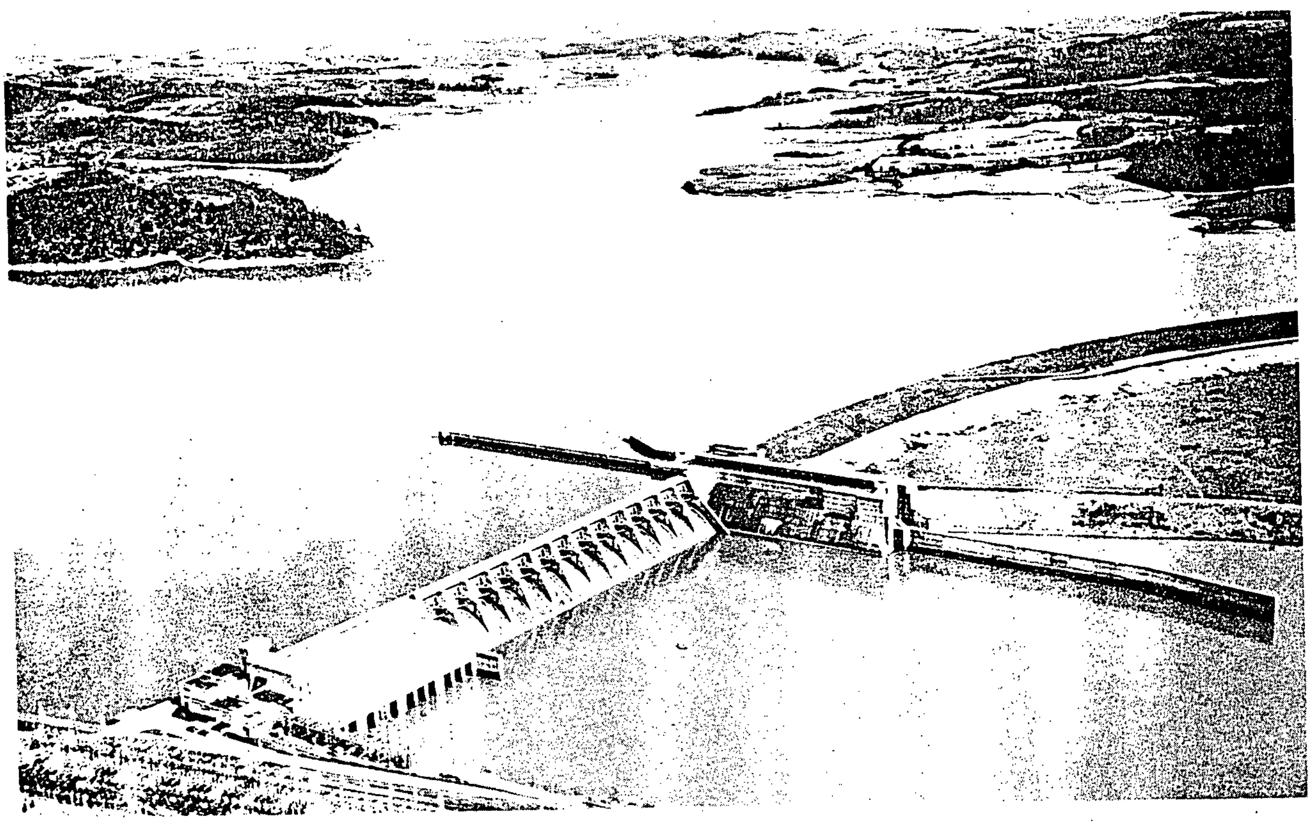


FIGURE 12.—*Fort Loudoun (in 1960 construction started on a bridge over the dam).*

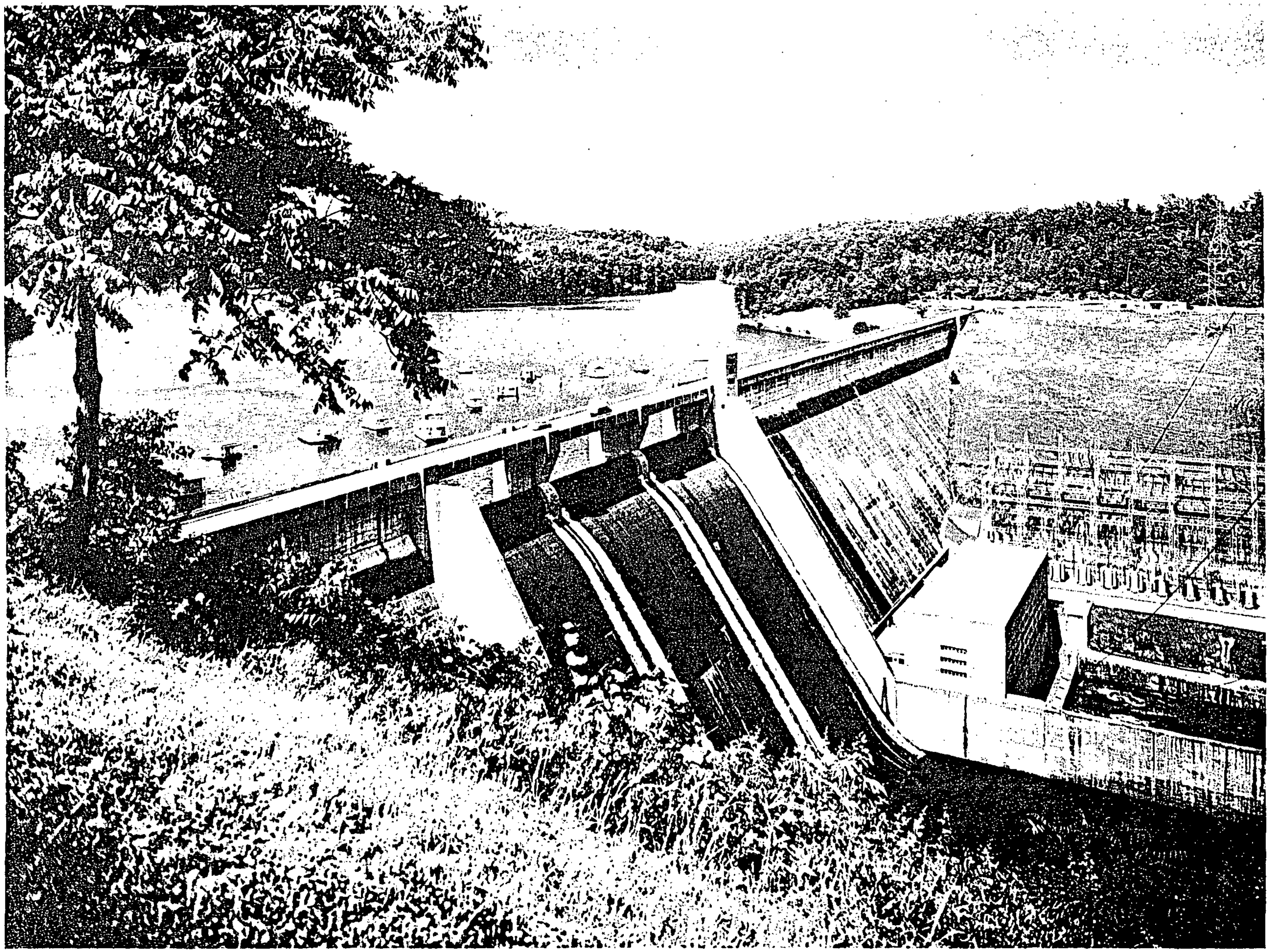


FIGURE 13.—*Norris—the first dam built by TVA.*

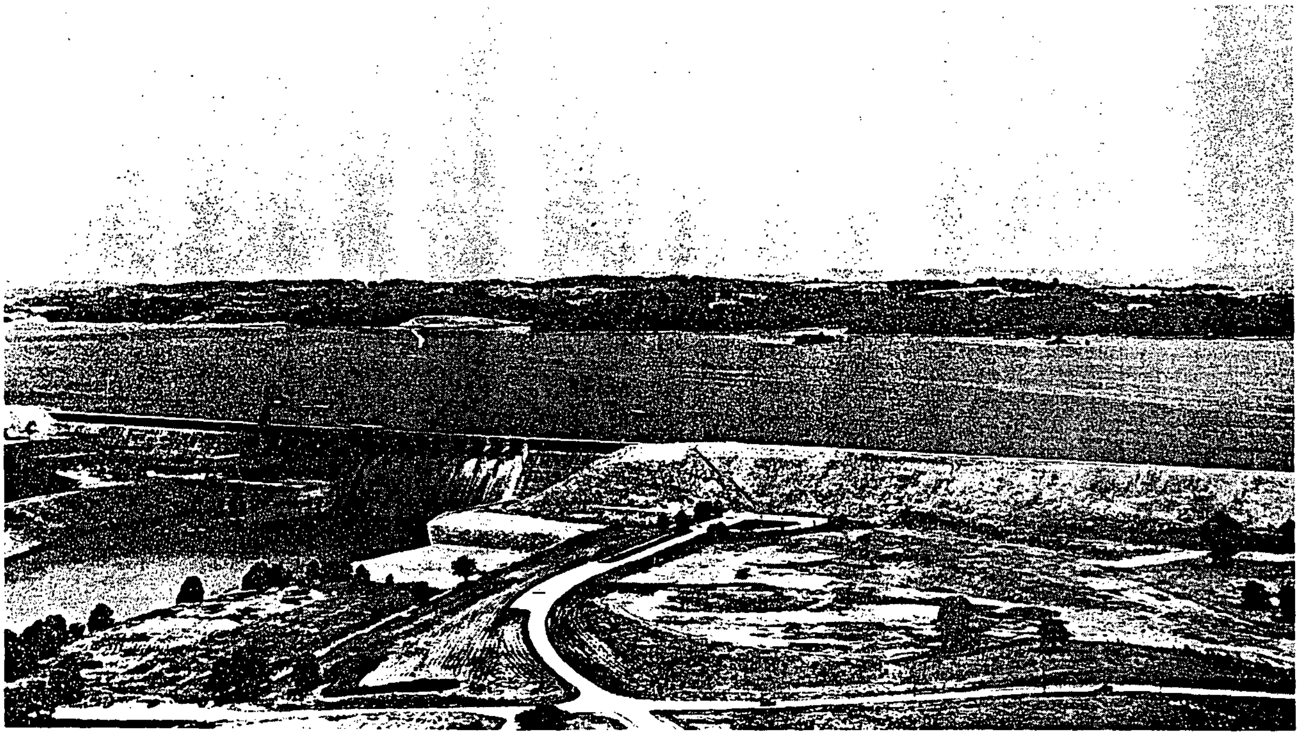


FIGURE 14.—*Cherokee.*

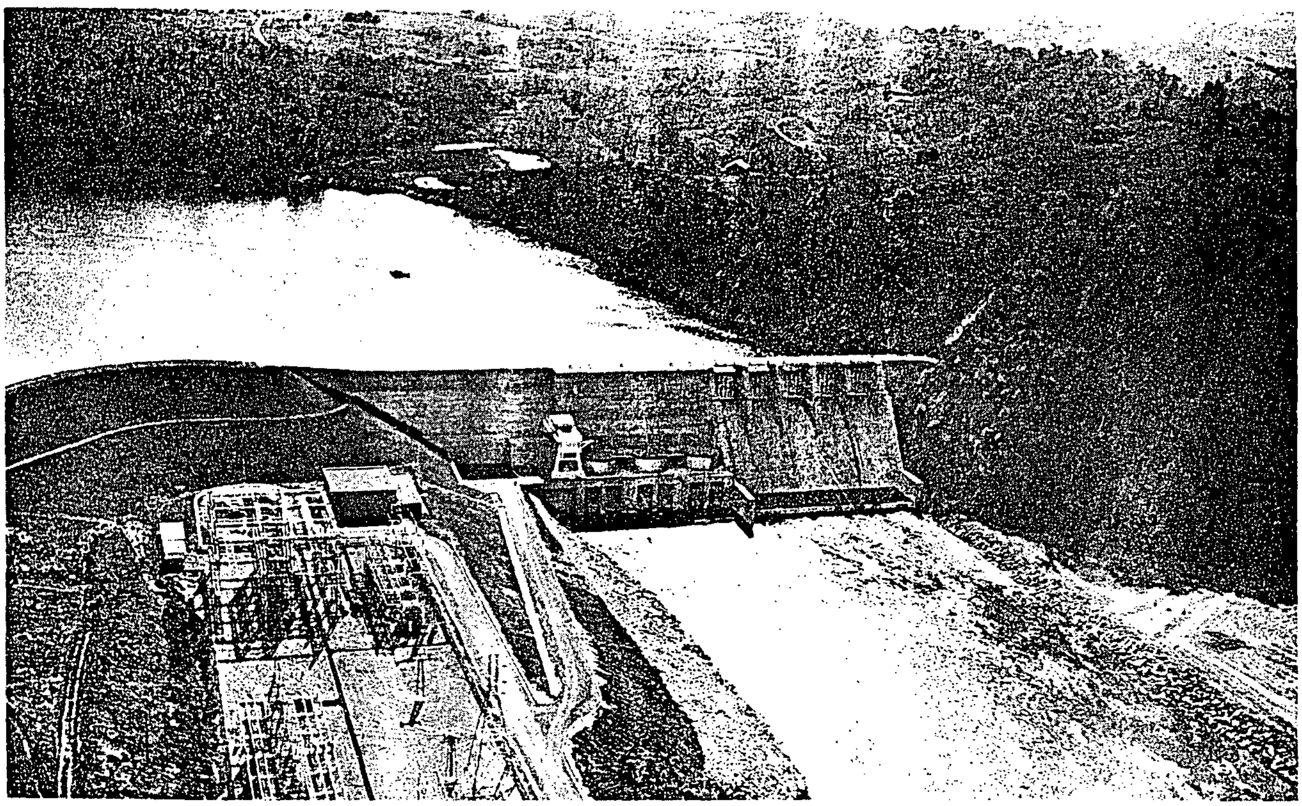


FIGURE 15.—*Boone.*

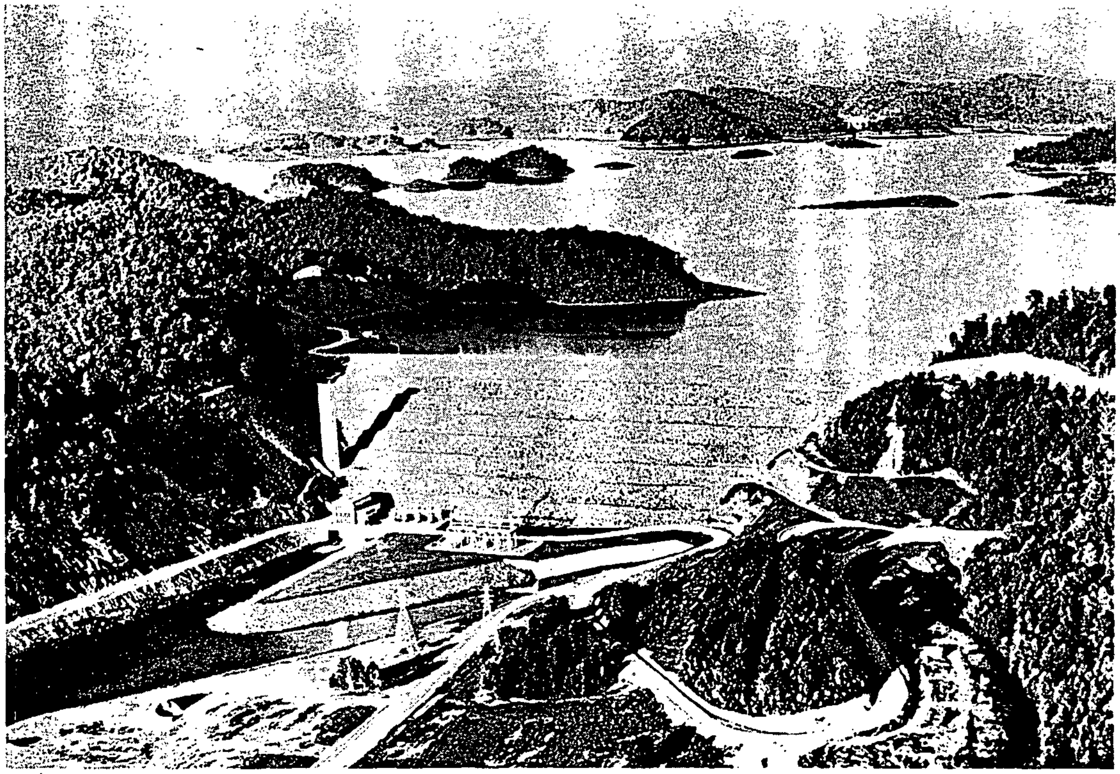


FIGURE 16.—*South Holston.*

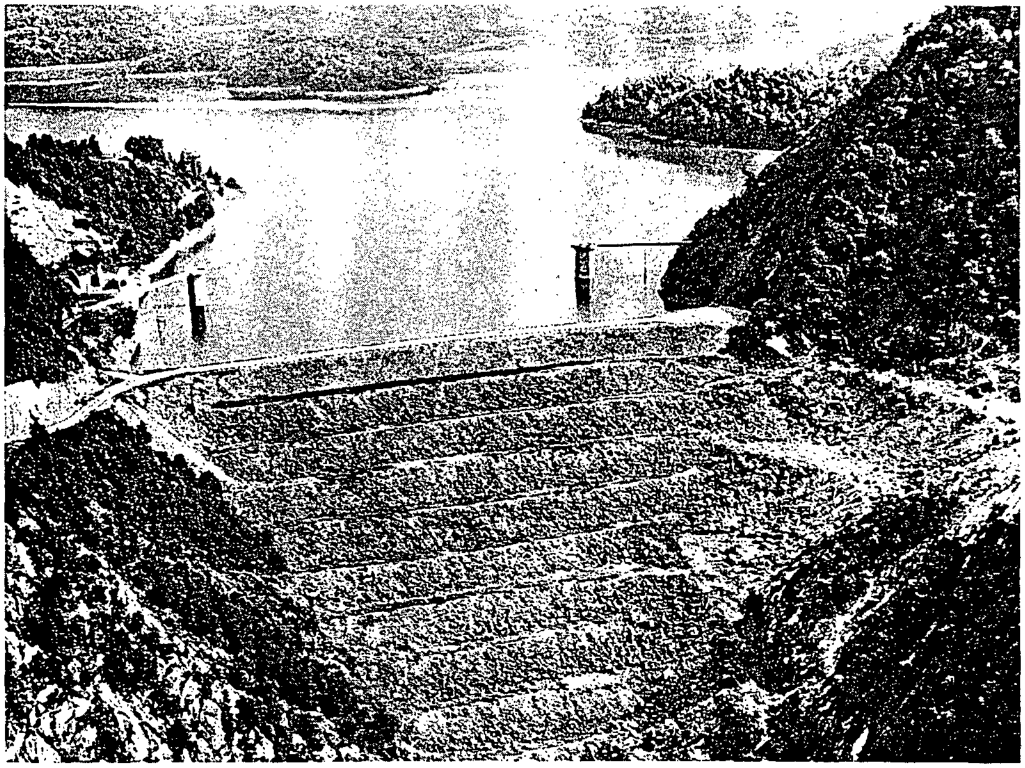


FIGURE 17.—*Watauga.*

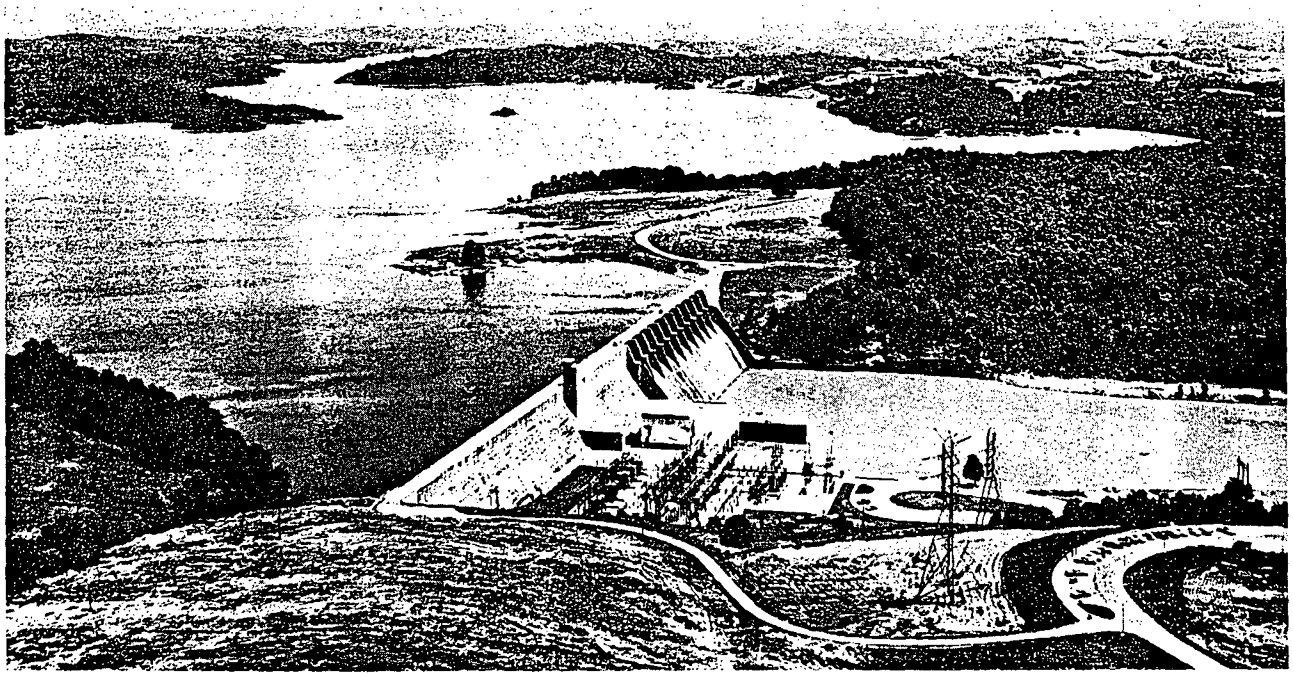


FIGURE 18.—*Douglas.*

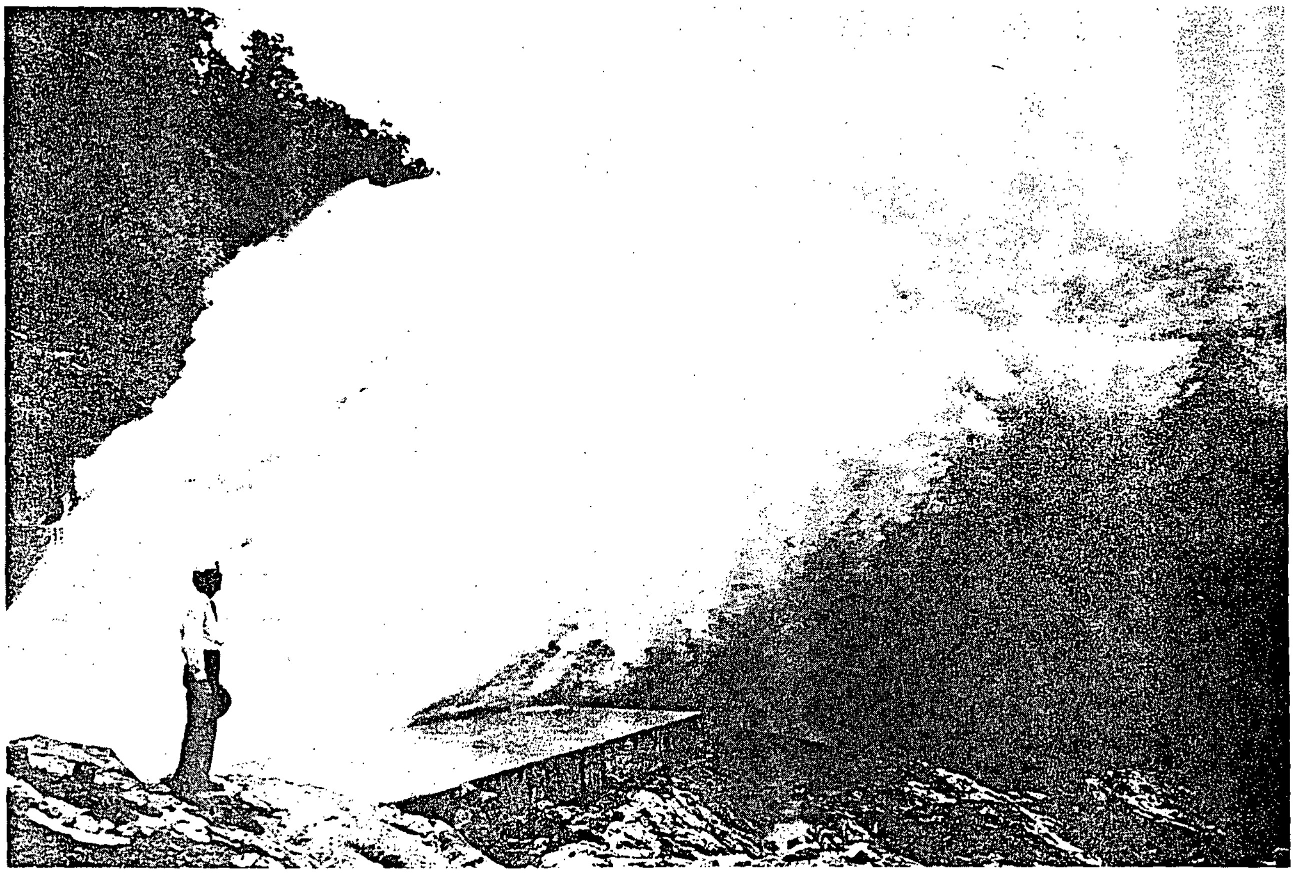


FIGURE 19.—*Fontana spillway discharging flood waters (see frontispiece facing page 1 for view of dam).*

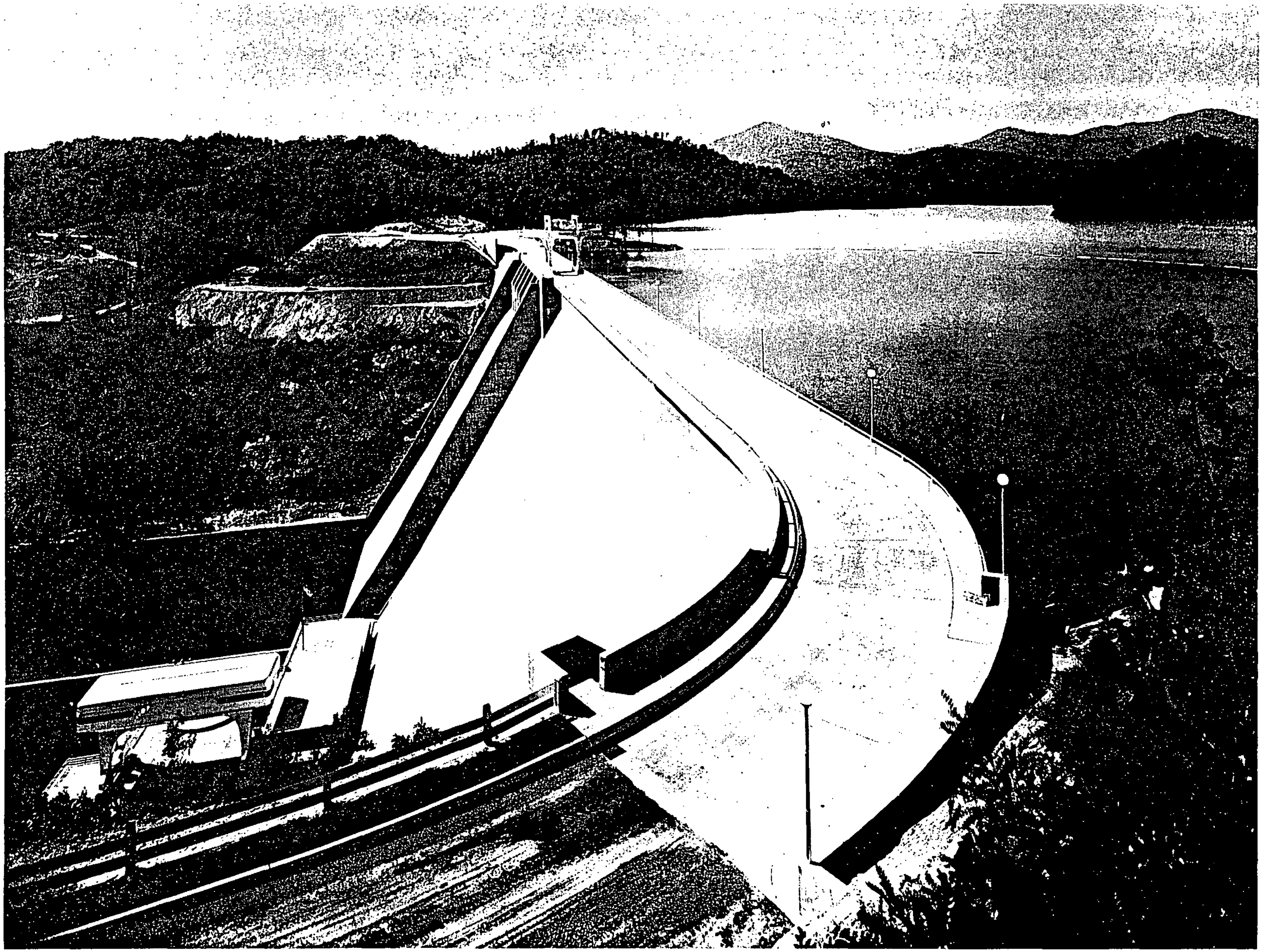


FIGURE 20.—*Hiwassee.*

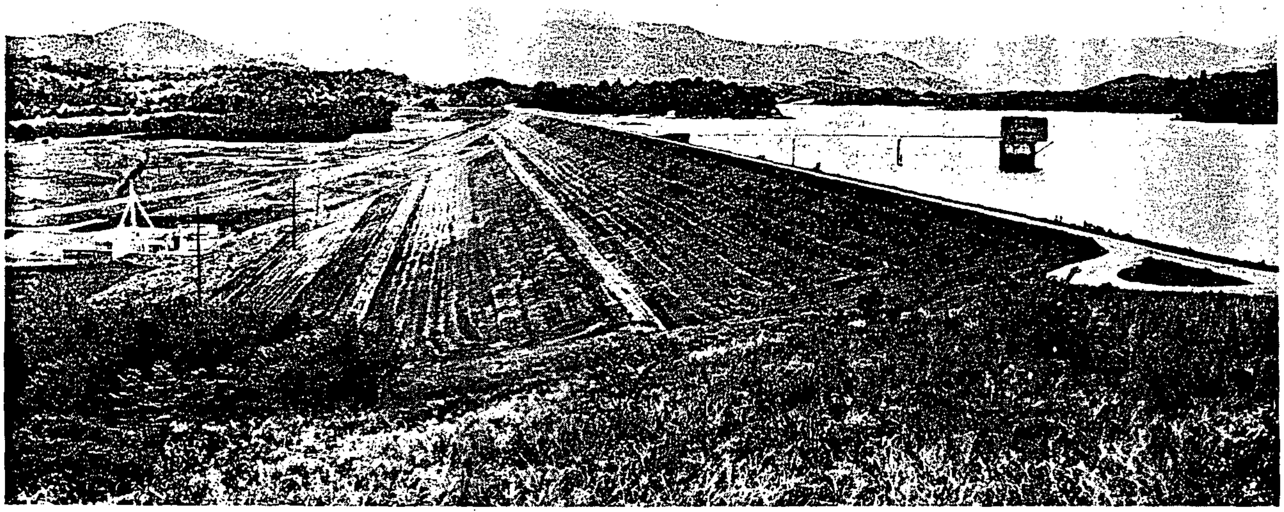


FIGURE 21.—*Chatuge.*

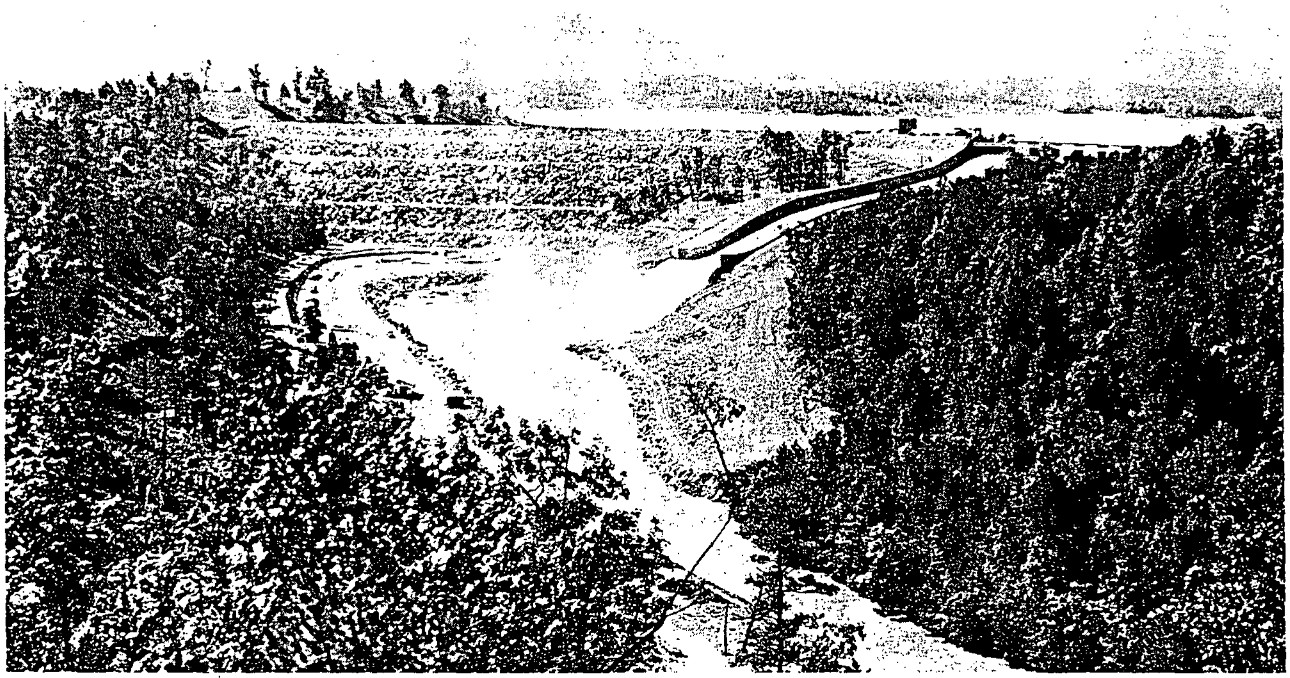


FIGURE 22.—*Nottely.*

FIGURE 23.—*The storm of March 1867 produced a record flood which destroyed this military bridge at Chattanooga (Hiener Photo Collection, Chattanooga).*

CHAPTER 2

FLOOD-PRODUCING STORMS

The geographical situation of the Tennessee River Basin within the temperate zone and in the southeastern part of the United States is most favorable to the occurrence of heavy and widespread storms. Major sources of moisture lie only a relatively short distance to the south in the Gulf of Mexico and the Caribbean Sea, prevailing winds in the region are from the south and west, and between these moisture sources and the Tennessee Valley there are no important barriers except the southern boundary of the Valley itself. The relation of the Valley to these major sources of moisture is shown by figure 24.

These heavy and widespread or flood-producing storms which occur over the Tennessee Valley are discussed in this chapter. It presents their principal characteristics in some detail, summarizes Valley-wide rainfall data, describes outstanding storms which produced floods prominent in the Valley's flood history, and briefly discusses intense storms over small areas. In conclusion, the chapter describes how observed storm data can be used to determine probable future extreme storms for a watershed. Such storms would produce floods of a magnitude for which estimated data would be suited for the design of water control and flood protection projects.

PRINCIPAL CHARACTERISTICS OF STORMS

Seasonal occurrence

Flood-producing storms can and do occur over tributary basins of the Tennessee River in every month of the year. The recorded experience of the last 90 years or more on the main river, however, shows that floods are more frequent, more widespread, and generally larger in the winter period from mid-December to mid-April than in any other season of the year. The record on figure 25 for the Tennessee River at Chattanooga illustrates this characteristic. Approximately 90 percent of the crest above flood stage of 30 feet at Chattanooga occurred in the December through April period. In the summer and fall months, the record shows that floods have never occurred at Chattanooga more than twice in any month and in three months there have been no floods in 90 years.

The same general pattern of flood occurrences prevails over most of the Tennessee Valley, even on drainage basins one-hundredth the size of that above Chattanooga. Principal exceptions to the rule occur on tributaries in the southeastern mountain region where torrential rains accompanying tropical hurricanes produce destructive floods in the summer and fall.

The greater incidence of winter floods results from a more frequent occurrence of heavy, persistent precipitation at the same time that vegetation is dormant and ground conditions favor a high rate of runoff. The same types of storms may occur in the summer and fall, and the moisture content of the inflowing air very likely will be greater in these warmer months than in the colder ones. However, the necessary combination of meteorological conditions to cause flood-producing rainfall occurs only rarely in summer. Also, surface conditions in summer are much less favorable to high rates of runoff. As a result, large floods over sizeable areas in the summer and fall are exceptional, and over the period of record—including historical accounts before the start of official records—no great Valley-wide flood has occurred during these seasons.

Source and paths of storms

The location of the Tennessee River Basin in the warm temperate zone exposes it to both continental and maritime polar air masses and to near extremes of tropical air. Modified polar air can and does cover the Basin at intervals throughout the year. Being relatively near the source region of tropical air in the Gulf of Mexico, the Basin is subjected to near maximum temperatures and moisture during periods of persistent northward flow of tropical air. Many major storms occur along and near the polar front that exists between air masses originating in the polar regions and the tropics.

The greater part of the annual precipitation in the Tennessee River Basin results from the passage of extra-tropical cyclones over or near the Basin. These cyclonic movements constitute the major source of precipitation during the winter months, but during the summer season fewer cyclonic rains occur and a higher proportion of convective showers prevails. The great storms, however, result from the less frequent, but more complex, synoptic situations

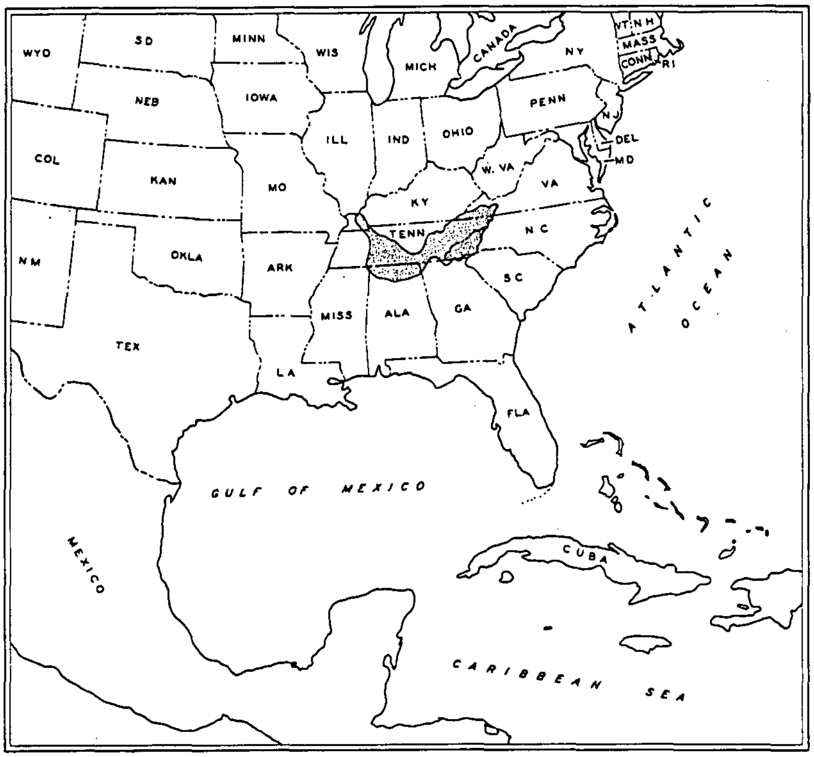


FIGURE 24.—*Relation of Tennessee Valley to major sources of moisture.*

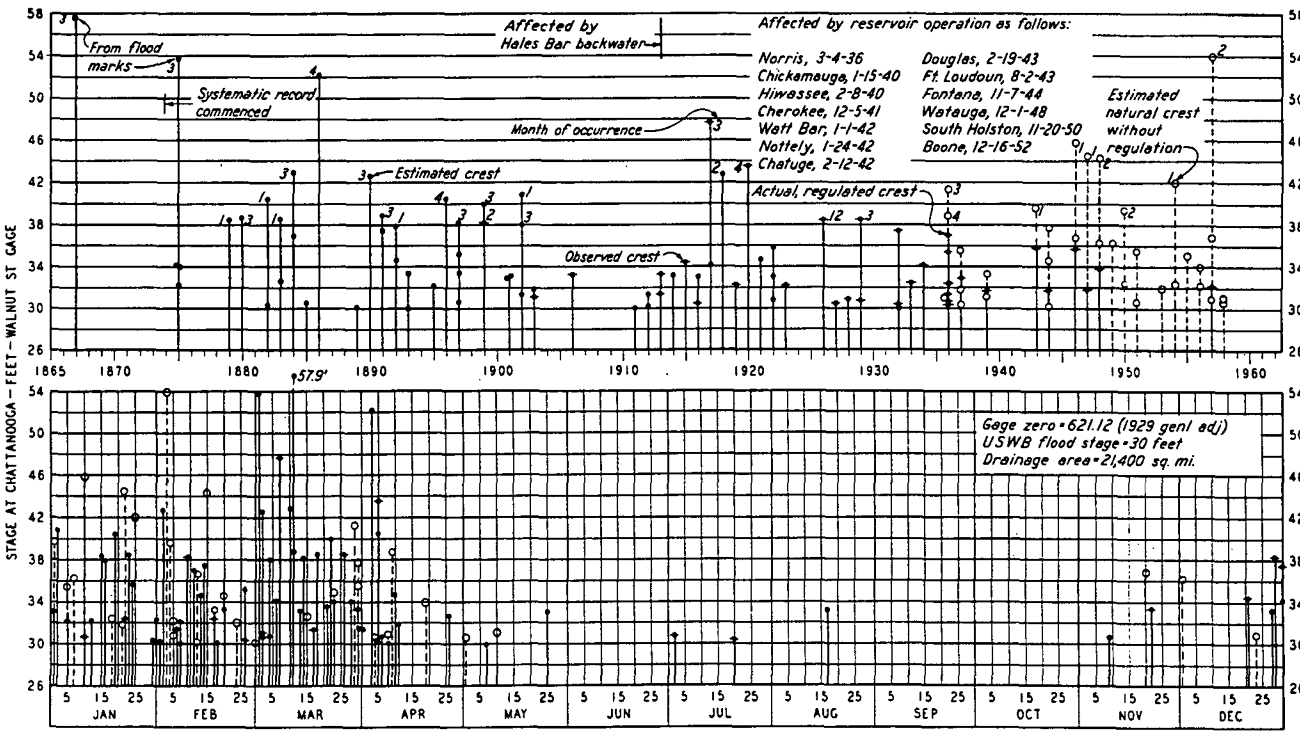


FIGURE 25.—*Yearly and seasonal occurrences of floods—Tennessee River at Chattanooga, Tennessee.*

classified by meteorologists as extra-tropical cyclones along quasi-stationary fronts, warm air covergences, and decadent West Indian hurricanes.

Storm types

Of the five types of flood-producing storms—as classified by the U. S. Weather Bureau in a report [1] covering the Ohio River Basin above Pittsburgh—only the first four types affect the Tennessee Basin. Actually, there is no marked distinction between types I, II, and III, for type I is a prerequisite to types II and III. An additional type which may cause destructive floods on small streams in the Valley, however, is the thunderstorm. In the following paragraphs types I through IV and thunderstorms are described and examples are cited.

Type I storms—This type of storm is a wave disturbance along a quasi-stationary front lying south of the Basin. It is one of two types of storms which can produce maximum or near maximum rainfall over the entire Basin. It occurs when the prevailing westerly winds aloft are strong, and the north-south fluctuations or waves on the polar front are of moderate amplitude. A circulation of this sort results in currents of air of marked contrasting heat and moisture content being brought close together, thereby providing a powerful source of energy. However, the prevalence of the strong westerlies causes a rapid translation of waves along the quasi-stationary front. The result is a relatively narrow band of intense precipitation of short duration. As this situation may persist for several days, a series of moderate rains occurs in bands that move progressively southward. Storms of this distinct type prominent in the flood history of the Tennessee River Basin are those of March 1-7, 1867; March 26-April 1, 1886; and February 11-15, 1948.

Type II storms—This type of storm occurs when deep warm moist tongues of air are subjected to convergence over the Basin. This convergence takes place when a deep cold air mass moves far southward of the Basin behind a low-pressure system which is moving northward or northeastward west of the Appalachian Range, forming the eastern boundary of the Basin. Due to the cyclonic circulation (counterclockwise in the Northern Hemisphere) of the cold air mass aloft, warm moist air is drawn northward in front of the cold air. In this hemisphere northward moving air normally curves anticyclonically (clockwise) due to the motion of the earth. However, if the cyclonic motion aloft is pronounced, it makes the northward moving warm air curve cyclonically. In so doing, the warm tongues of air moving northward are subjected to convergence. If the warm air is heavily charged with moisture and has a tendency to be unstable, this convergence will render it actually unstable. The result of this condition is usually intense convection, producing high amounts of rainfall. The storm of August 29-30, 1940, is an excellent example of this type of storm which produced flooding over a wide area in the eastern part of the Basin.

Type III storms—This type consists of deep, occluded low-pressure centers which stagnate over or immediately south of the Basin. It is similar to type II as far as upper air circulation is concerned, but the cold air mass pushes farther southward west of the Basin, causing a low-pressure center to form and occlusion to take place over the central United States, with very little if any movement during the process. As the deep cyclonic currents of cold upper air rotate about the surface low-pressure center, the low becomes more pronounced or deepens, developing an almost vertical axis. Such lows can move in almost any direction, but generally move northward and do so very slowly. The intense low pressure centers cause strong currents of warm moist air to move northward over the Basin. Rainfall usually occurs in the early occluding stages when an upper cold front swings eastward ahead of the surface low. Convergent flow precedes the upper cold front, causing a narrow band of moderate rainfall to move rapidly eastward across the Basin. Clear cut examples of this type of storm are relatively rare, and it is usually associated with type I or type II storms.

Type IV storms—These storms are the decadent tropical storms which originate as West Indian hurricanes and move inland. Such storms carry deep unstable moist air up to very high levels and dissipate rapidly as they leave the water areas due to the rapid consumption of energy associated with increased surface friction over land areas. As this type moves inland, it assumes extra-tropical cyclonic (low-pressure system) characteristics, or it may move up an old pressure trough, resulting frequently in moderate to heavy precipitation over large areas. In crossing mountain barriers such as the eastern divide of the Basin or the Great Smoky Mountains within the Basin, these storms produce torrential rainfall because of the increased vertical motion caused by the mountains. The storms of July 14-16, 1916, and August 11-16, 1940, are examples of this type affecting sizable areas in the eastern part of the Tennessee Valley.

Thunderstorms—There is an additional type of storm which occurs frequently in the Basin during the summer months and which may cause destructive floods on small streams. This is the thunderstorm, a convective precipitation[2] resulting from the rising of air warmer than its surroundings. The air

1. U. S. Weather Bureau and Corps of Engineers, *Maximum Possible Precipitation Over the Ohio River Basin Above Pittsburgh, Pennsylvania* (Vicksburg: U. S. Waterways Experiment Station, June 1941), pp. 39-62.

2. R. K. Linsley, Jr., Max A. Kohler, Joseph L. H. Paulhus, *Applied Hydrology* (New York: McGraw-Hill Book Company, 1949), pp. 65.

continues to rise because of its lesser density until it reaches a level or stratum where it has the temperature of its surroundings. The convection is caused either by heating of surface air, cooling of the upper air, or mechanical lifting over a frontal surface or mountain. These storms usually cover small or local areas with very intense rainfall of showery nature. The total runoff is seldom of such magnitude as to affect the larger streams of the Basin. However, large floods have resulted on smaller streams from thunderstorms. An example is the storm of June 18, 1939, near Lewisburg, Tennessee, when up to 9 inches of rain fell in 3 hours, causing record floods on small tributaries of the Duck and Elk Rivers.

Orographic influence on storm rainfall

The mountains of the eastern portion of the Basin exert an important influence upon the storm rainfall and the total annual precipitation of that section. During cyclonic storms, the orographic influence adds substantially to the rainfall produced by the storms, and high elevation gages may record rainfall amounts several times that at the lower levels. The mountains also supply the lift to set off convective precipitation so that rainfall in the mountains occurs more frequently than in the valleys.

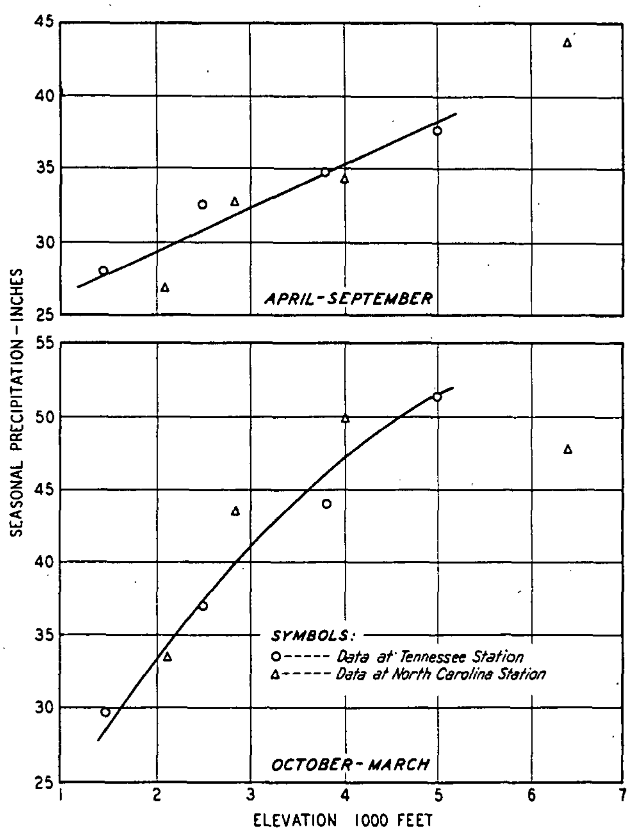


FIGURE 26.—*Orographic influence of Great Smokies on precipitation.*

Figure 26 shows the increase of seasonal precipitation with altitude as demonstrated by observations in the Great Smoky Mountains. These data were collected cooperatively by the TVA, the U. S. Weather Bureau, and the National Park Service during the 5-year period 1946-1950.[1] The effect is more marked in the October to March season of general cyclonic storms than in the April-September period of convective showers.

Figure 27 shows the apparent orographic influence exerted by the Southern Appalachian Mountains during the storm of August 11-16, 1940. This was a tropical storm of hurricane intensity which moved inland over Georgia and South Carolina and followed an unusual curving path which crossed Tennessee from Chattanooga to Bristol and thence moved eastward and southward to Greensboro, North Carolina. As the storm approached and moved parallel to the Southern Appalachian Mountains, on August 12 and 13, very heavy rainfall occurred, particularly on the windward side of the mountains.

Frequency and duration of storms

A comprehensive investigation of maximum observed storms and storm types yielded valuable information on the frequency of occurrence and duration of the various types of flood-producing storms. In this study of past storms to determine the maximum types for the Tennessee River Basin and the most critical examples of each type, the search was not restricted to those storms which have occurred immediately over this area. It was possible to define a much larger area than the Tennessee River Basin which is still meteorologically homogeneous. Storms in this larger region could have occurred anywhere within that area, including the Tennessee River Basin, within the limits of chance distribution of the cyclonic forces present at that time. This broadening of the area of search for observed storms provided a much greater source of data. The area of homogeneity considered was approximately 600 miles from north to south and 900 miles from east to west and included the states of Alabama, Arkansas, Georgia, Kentucky, Mississippi, North Carolina, South Carolina, and the Great Valley section of Virginia, as well as Tennessee.

The preliminary list of observed storms developed in this investigation contained the approximate dates and locations of 1,411 storms. As thunderstorms are seldom widespread, this type of storm was not included in the preliminary list. Further review was made to reduce the list to a more workable number of the hydrologically most important storms which could be subjected to meteorological analysis to determine the storm types present. The selection of these important storms was accomplished through an inspection of the preliminary isohyetal maps. During this selection, the

1. TVA, *Precipitation in Tennessee River Basin Annual 1949* (Knoxville, 1950), pp. 14-17.

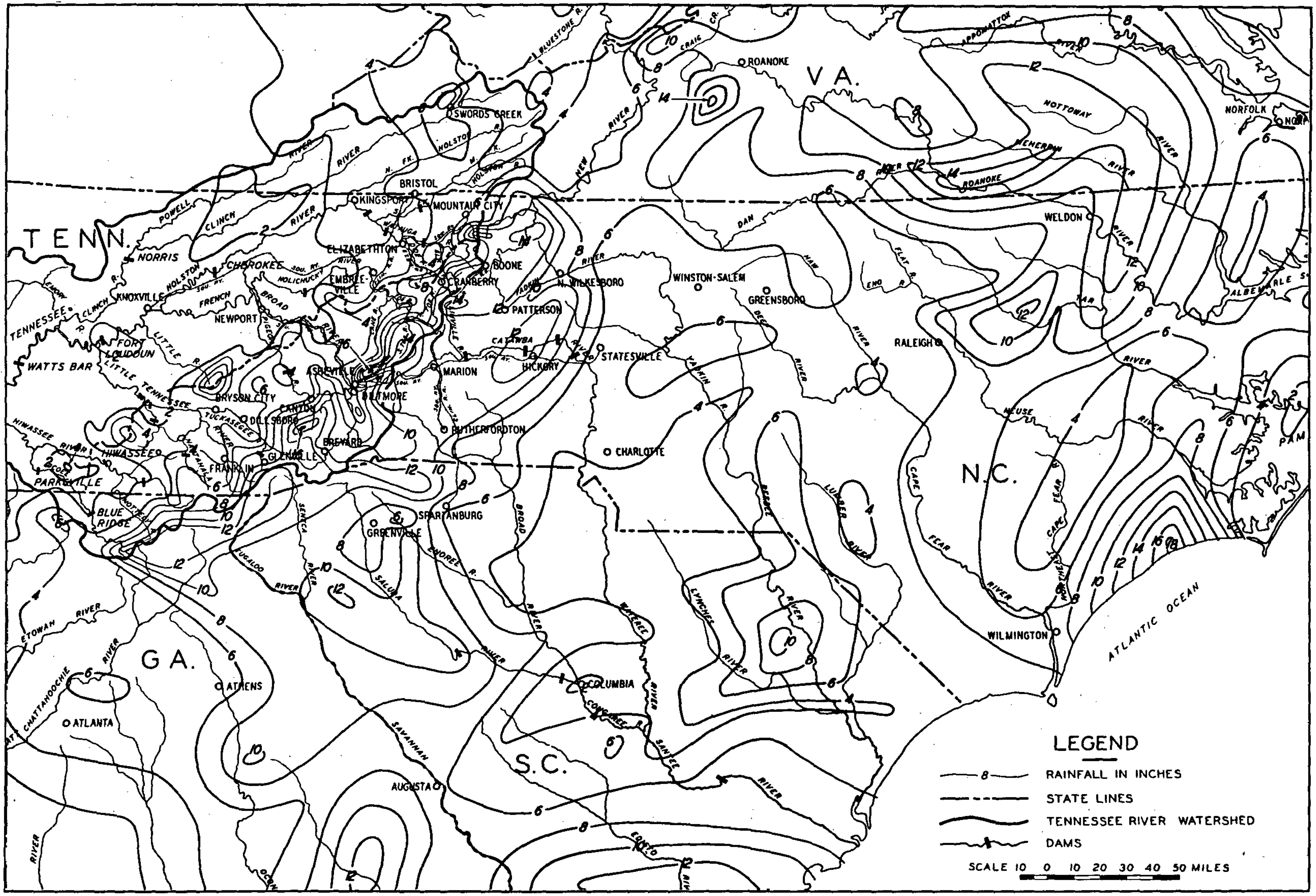


FIGURE 27.—*Orographic influence of Southern Appalachian Mountains in mid-August 1940 storm.*

area of homogeneity was further restricted, eliminating those storms occurring entirely in the southern half of the Gulf states and the eastern half of the Atlantic states. This procedure prevented the inclusion of a disproportionately large number of purely coastal storms which, because of their excessive amounts, might have dominated the finally selected groups of storms considered representative of the types affecting the Tennessee River Basin region. The resulting list numbered 167 important storms.

The frequency of occurrence of the 167 storms selected is a fair sample of the monthly distribution of storm types in the Basin. Minor irregularities of the data are undoubtedly due to the chance occurrence of a greater or fewer number of storms in a particular month, but the broad trends are of interest and importance. The distribution of the 167 storms according to type is presented in figure 28. In this chart types II and III storms have been combined since many severe storms starting as type III finally occluded as type II. The important type I or quasi-stationary frontal storms occur during all months of the year, but their frequency is greater during the months from December through May when the polar front is near the Tennessee River Basin region. The types II and III storms which are due to warm sector convergence occur during all months of the year, having greatest frequency in July. The low frequency of convergence type storms during February is attributed to the normal position of the polar front which averages its farthest southerly extension during that month. Convergence rains of magnitude occur in the tropical air to the south of the polar front, hence, their frequency of occurrence

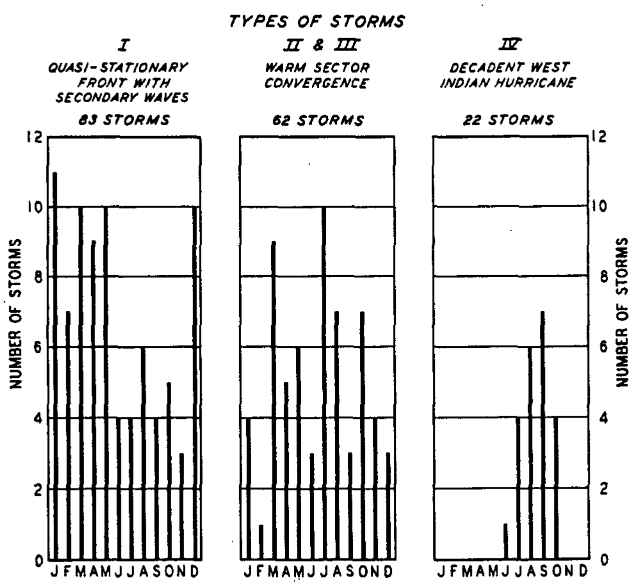


FIGURE 28.—*Monthly distribution of storm types affecting Tennessee Valley.*

in February is low. The type IV storms or decadent West Indian hurricanes affect the Tennessee River Basin principally from June to October with the greatest frequency during August and September.

In summary, from the data available on these 167 storms it is evident that quasi-stationary and convergence maximum storms affect portions of the Tennessee River Basin during all months of the year with varying frequency. Important and great storms of hurricane origin occur principally from June to October.

From a review of the records of floods throughout the Basin, it is apparent that the frequency of occurrence of flood-producing storms and their duration will vary with the size of the drainage area. The size of the area affects the time of concentration of runoff which is the time required for water to flow from the remotest point in the area to the outlet. As thunderstorms occur frequently and throughout the year over the Tennessee River Basin, small areas having short times of concentration are subjected to more frequent flooding than larger areas. Large areas seldom experience widespread floods from thunderstorms. In general, a flood-producing storm on an area within the Basin occurs on the average of about once every year or two and usually in the period December through April, with March having the highest frequency. However, July approaches the winter and spring months in frequency of flood-producing storm occurrences in some areas.

Depending upon the storm type, the duration of flood-producing storms may vary from a matter of one or more hours for thunderstorms to as much as a week or longer for frontal storms.

Sequence of storms

In many instances the outstanding floods on the Tennessee and Ohio Rivers have resulted from the cumulative effect of an extended sequence of storms. The greatest flood of record on the lower Tennessee River resulted from rains spread over 21 days in March 1897. A similar sequence of storm events occurred in January 1937, culminating in the disastrous record-breaking flood that swept the Ohio River valley during the latter part of January and early February. Such a sequence of events has been observed in lesser degrees of severity and duration several times in the period of record, and a recurrence must be considered as highly probable in any study of storm replication.

Based on a thorough study of storm intervals, the Hydrometeorological Section of the U. S. Weather Bureau concludes that the intensity of the maximum possible storm is such that its termination precludes the development of appreciable rains within less than three days.[1] While it may be unreasonable to assume that the maximum possible rainfall will

1. *A Preliminary Report on the Maximum Possible Precipitation Over the Potomac and Rappahannock River Basin*, Hydrometeorological Section, USWB, in cooperation with Corps of Engineers, U. S. Army, July 24, 1943, p. 83.

be followed by a storm of equal or nearly equal severity, this circumstance is a possibility. An unusual series of severe storms in close sequence was observed in July 1916 when three hurricanes occurred within a total period of 16 days. Only two days elapsed between the rains associated with the hurricane rains of July 14-16 and July 18-21. In August 1928, only three days elapsed between storms. It is certainly reasonable to expect that the maximum rain can be preceded by showers and followed by appreciable rain after a three-day interval.

ANNUAL AND MONTHLY RAINFALL

There is a wide variation in the precipitation which annually falls over the Basin. The 70-year mean annual precipitation over the Tennessee Valley for the period 1890-1959 is 51.51 inches. The monthly and annual average amounts east and west of Chattanooga and over the entire Basin during that period are given in table 2.

TABLE 2.—*Average precipitation—Tennessee River Basin.*

Month	Average precipitation 1890-1959—Inches: East of Chattanooga	West of Chattanooga	Entire Basin
January	4.57	5.29	4.91
February	4.67	4.98	4.81
March	5.30	5.76	5.51
April	4.19	4.78	4.47
May	4.10	4.18	4.14
June	4.44	3.98	4.23
July	5.28	4.51	4.92
August	4.50	3.82	4.18
September	3.26	3.11	3.19
October	2.89	2.80	2.85
November	3.24	3.88	3.54
December	4.55	5.00	4.76
Annual total	50.99	52.09	51.51

The wettest of the 70 calendar years over the entire Basin was 1957 with an average of 64.6 inches. The period between November 1948 and October 1949 was the wettest 12-month period ever measured in the Basin, averaging over 69 inches for the whole area. The driest calendar year was 1941 with an average of 37.9 inches. Figure 29 is an isohyetal map of mean annual precipitation for the Basin based on the 25-year period 1935-1959. The mean precipitation for this period is very nearly the same as for the 70 years for which data of varying completeness are available.

The greatest extremes in annual precipitation occur in the eastern half of the Basin where heavy rain falls on the high levels of the Blue Ridge and Great Smoky Mountains and relatively light rain falls in the shielded valleys to the north of the ridges. The average annual precipitation exceeds 80 inches at several stations located in the mountains of the French Broad, Hiwassee, and Little Tennessee River watersheds. The maximum station average is 93.9 inches for the period 1936-1959 at Coweeta No. 8, North Carolina, in the headwaters of the Little Tennessee River. In contrast, several stations located in the French Broad and Holston River watersheds have annual averages of less than 40 inches. The minimum station average is 37.0 inches for the period 1946-1959 at Weaverville, North Carolina, in the French Broad River valley north of Asheville.

The highest total precipitation in a calendar year at an individual station was 133.3 inches at Haywood Gap, North Carolina, in 1957. During the 12-month period November 1948 through October 1949, a total of 145.5 inches was recorded at Coweeta No. 8. The minimum recorded annual precipitation since TVA began collecting precipitation data was 18.7 inches measured in 1941 at Colesville, Tennessee, located in the upper Holston River watershed. In the same year, the rain gage on Mt. Mitchell in the Blue Ridge Mountains of North Carolina recorded 19.3 inches of rainfall in the month of July alone.

March with an average of 5½ inches is the month of heaviest precipitation over the Basin. In the portion of the Basin east of Chattanooga, July is almost as wet as March while in the western portion January ranks second to March with July in sixth place. The driest month in the Basin is October with an average under 3 inches.

The monthly and annual variation of the mean precipitation throughout the Basin is shown in table 3 by precipitation statistics for selected long-term stations representative of the major subdivisions of the Basin. These are Elizabethton, Tennessee, in

TABLE 3.—*Mean monthly precipitation in inches for selected stations for period of record through 1959.*[1]

Station	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
Elizabethton	3.47	3.40	4.22[2]	3.39	4.09	4.71	5.16[3]	4.41	2.90	2.45	2.36	3.33	43.89
Hendersonville	4.71	4.63	5.36[2]	4.35	4.46	5.21	6.19[3]	5.80	4.47	4.08	3.34	5.16	57.76
Knoxville	4.71	4.74	5.08[2]	4.26	3.77	4.10	4.62[3]	3.85	2.84	2.51	3.36	4.35	48.19
Murphy	5.60	5.69	6.23[2]	4.82	4.04	4.97	5.68[3]	4.93	3.28	3.06	3.80	5.23	57.33
Chattanooga	5.30	5.03	5.81[2]	4.68	3.89	4.03	4.60[3]	3.83	3.13	2.98	3.60	5.06	51.94
Lewisburg	5.34	5.18	5.85[2]	4.53	4.03	3.97	4.35[3]	3.97	3.18	2.89	3.81	4.86	51.96
Muscle Shoals	5.21	5.14	5.74[2]	4.62	3.90	4.05	4.53[3]	3.75	2.97	2.62	3.51	4.85	50.89
Johnsonville	5.58[2]	4.34	5.22	4.68	4.29[3]	4.04	4.12	3.73	3.45	2.87	4.17	4.52	51.01

1. Source: *TVA, Precipitation in Tennessee River Basin—Annual 1959,* pp. 11-13.
2. Maximum in winter period, November-April.
3. Maximum in summer period, May-October.

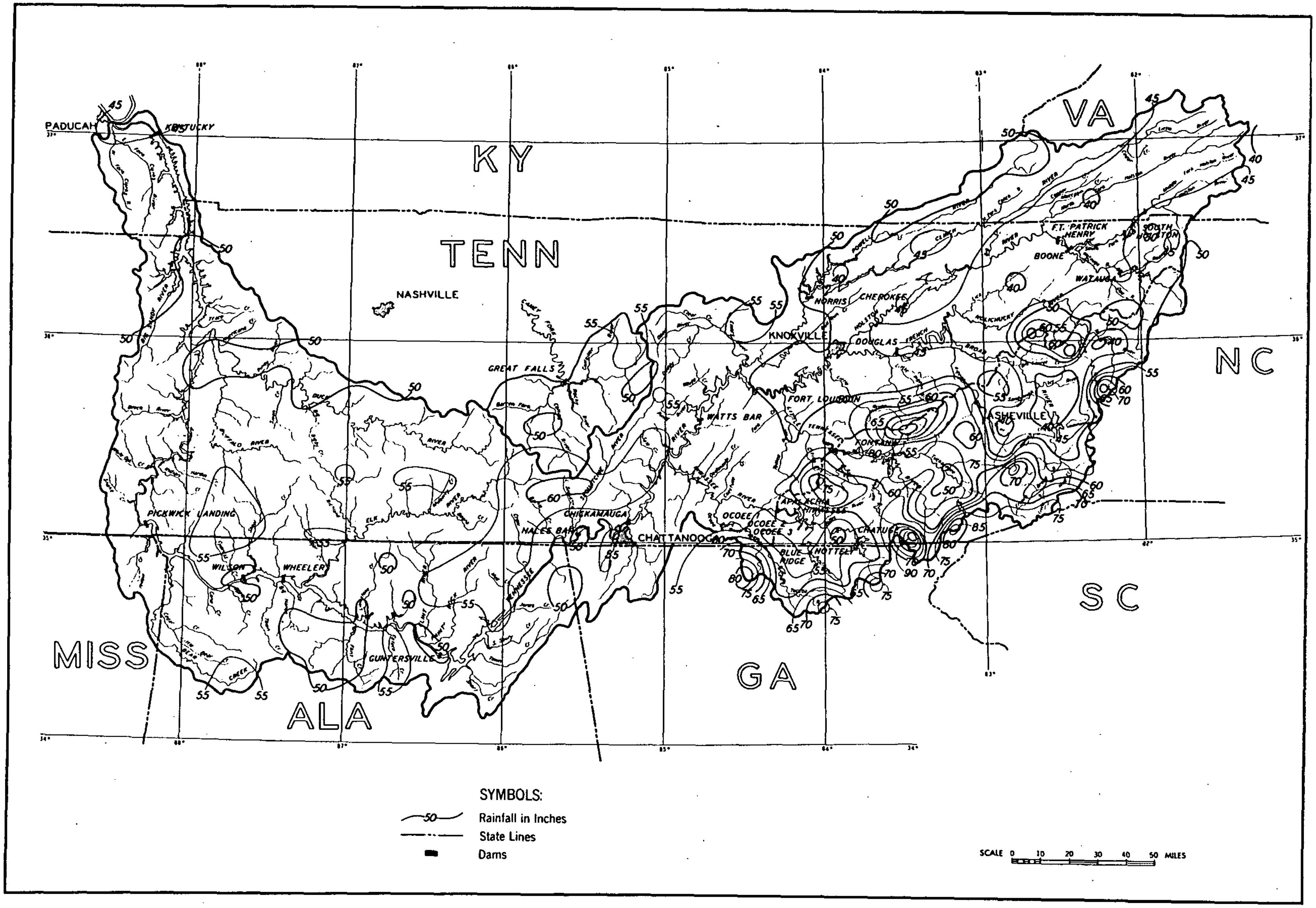


FIGURE 29.—*Mean annual precipitation—Tennessee River Basin—1935-1959.*

the northeastern section; Hendersonville, North Carolina, in the southeastern section; Knoxville, Tennessee, the east central section; Murphy, North Carolina, also in the southeastern section; Chattanooga, Tennessee, the central section; Lewisburg, Tennessee, the west central section; Muscle Shoals, Alabama, the southwestern section; and Johnsonville, Tennessee, the northwestern section. All have records of over 60 years. A study of this tabulation shows, with the exception of the extreme western section, that the maximum precipitation in the winter months November through April occurs in March, with a maximum in July during the summer months May through October.

Monthly rainfall in excess of 15 inches may occur occasionally in any part of the Basin, and monthly totals of over 30 inches have been recorded in the extreme headwaters of the French Broad and Little Tennessee Rivers. The heaviest precipitation of record in this area occurred in July 1916. In this month, North Carolina stations reported totals as follows: Highlands, 35.5 inches; Altapass, 35.4 inches; and Rockhouse, 36.4 inches. The Altapass station reported 22.22 inches in a 24-hour period on July 15-16, 1916, during the passage of a tropical hurricane. This was the greatest 24-hour amount ever recorded in the Basin. In contrast, monthly precipitation totals of less than 0.1 inch have occurred in the Basin, usually in September and October. Several long-term stations, largely in the western part of the Basin, have experienced entire months without measurable rainfall.

STORMS IN TENNESSEE VALLEY REGION

In the flood history of the Tennessee River and its tributaries a number of storm dates have figured prominently In the following section, some of these outstanding storms are described briefly. Some of the storms discussed were widespread and caused great floods along the Tennessee River and especially at Chattanooga. Included in this category are the storms and floods of 1867, 1875, 1886, 1897, and 1957. Other storms caused record or near-record floods over major tributaries, such as the storms of 1791, 1826, 1840, 1901, and 1940 in the east and southeast, the storm of 1929 on the Cumberland Plateau, and the storms of 1902 and 1948 on the western part of the Valley.

The storms are separated into three chronological groupings: (1) The early storms, or those occurring between 1791 and the beginning of systematic streamflow records in about 1875; (2) storms occurring between 1875 and the beginning of TVA flood control operations in 1936; and (3) storms during the period of TVA flood control operations, or from 1937 to date.

Early storms

Information relative to floods occurring prior to the establishment of stream flow records is fragmentary and lacking in detail. This is particularly true for the lesser floods which caused little or no damage. The occurrences of large floods of the past still linger in the minds of living persons who either saw or heard of them. Occasionally references to these larger floods are found in newspapers and other printed records.

1791—The flood of April 1791 is one of the earliest known in the Tennessee Valley region. Four separate references to this flood on the Swannanoa River in the French Broad River Basin were found in a flood history investigation of that river. At least two of these references are independent of the others, strengthening the facts supporting the flood and its magnitude. The weight of evidence regarding this old flood indicates that it probably was about 5 feet higher on the Swannanoa River near Biltmore than the flood of July 1916, the greatest in recent years in the French Broad River Basin. Old Knoxville newspaper accounts describe the 1791 flood as being very high also in the upper reaches of the Tennessee River.

1826—The highest flood of knowledge in the vicinity of Clinton, Tennessee, on the Clinch River occurred in March 1826. The flood height is supported by sufficient data to authenticate it within reasonable limits. From flood marks recorded in the vicinity and interviews with residents who had been told of the 1826 flood by older residents, it has been established that this flood was about 18 feet above the present flood stage at Clinton. High water data for various floods developed by the Corps of Engineers show that a great flood also occurred in March 1826 on the Cumberland River.

1840—According to stories told by old settlers and handed down through generations, one of the greatest floods in the Little Tennessee Basin occurred in May 1840. Four flood marks found for this flood on the Tuckasegee River, the principal tributary of the Little Tennessee River, indicate that most of the water probably came from that stream.

1847—According to the *Knoxville Register* of March 17, 1847, large floods occurred over the Clinch, Holston, and French Broad River Basins just prior to that date. At Knoxville, the crest stage was reported by the newspaper to be about 3 feet higher than the flood of 1826. The newspaper also states that the flood of 1791 was the only flood within the memory of the oldest inhabitant that exceeded the one in March 1847 at Knoxville.

Destructive floods in mid-December 1847 were reported in middle Tennessee, affecting particularly the Cumberland and Caney Fork Rivers. Newspaper accounts indicate the storm was widespread enough

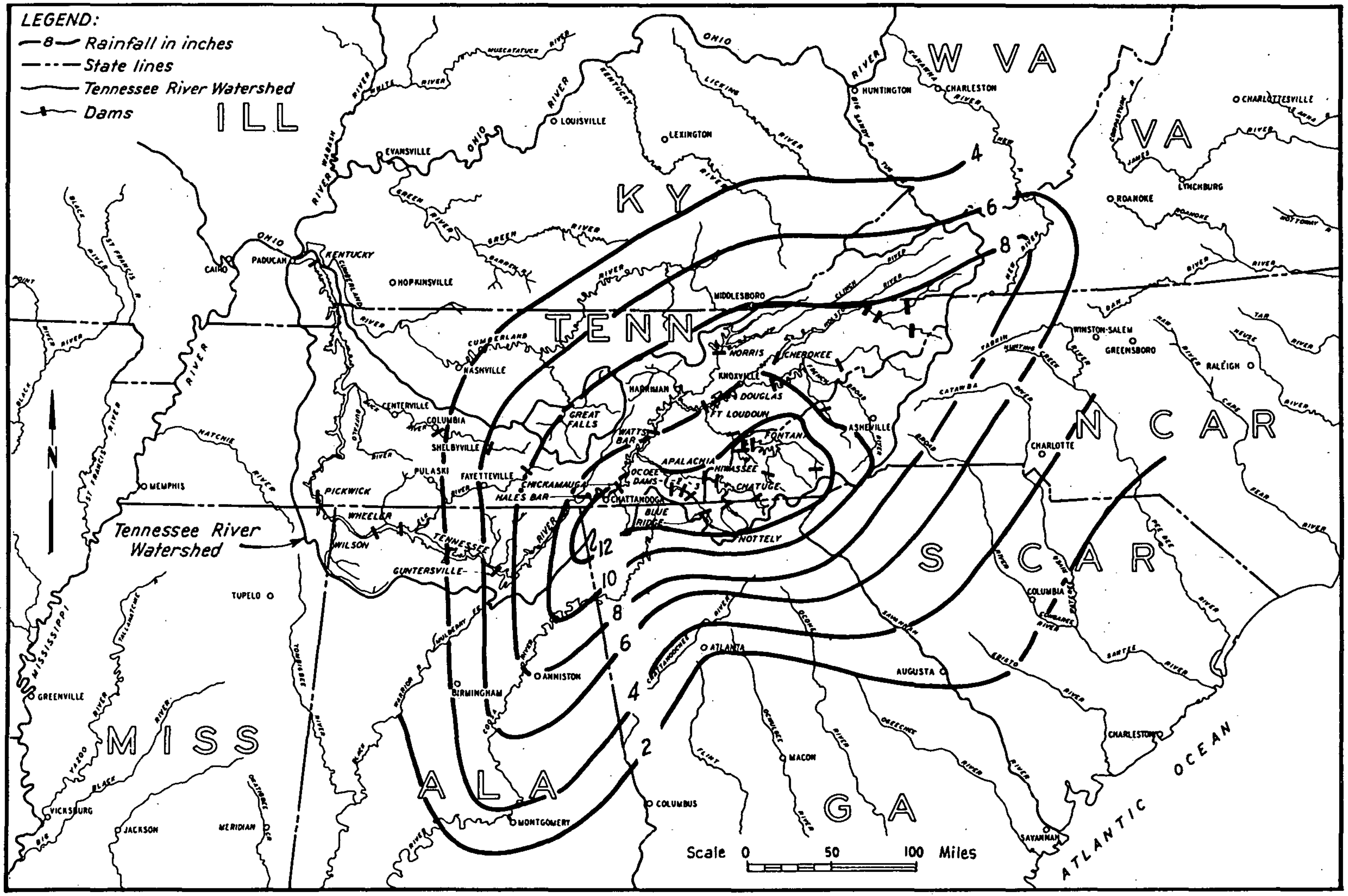


FIGURE 30.—*Estimated total rainfall—storm of March 1-7, 1867.*

to cause disastrous flooding of Cincinnati by the Ohio River.

1852—The book *Asheville and Buncombe County,* by F. A. Sondley, LL.D., contains some information on floods in the French Broad River Basin. A large flood, reported in this history as having occurred on August 28-30, 1852, did considerable damage in the French Broad and Swannanoa River Basins, washing away bridges and crops. The rain, according to the *Asheville News* of September 2, 1852, fell from 10:00 p.m. Thursday night, August 28, to sometime Friday night, August 29, without cessation.

1867—The storm of March 1-7, 1867, was one of the outstanding widespread storms in the Valley. This was a quasi-stationary frontal type with secondary waves. The storm is of historical importance in the Tennessee River Basin because the floods resulting from it are among the largest of record over a considerable portion of the area. On the Tennessee River at Chattanooga, where the crest stage was 57.9 feet, as well as on the lower portions of the Holston, French Broad, and Little Tennessee Rivers, the 1867 flood was the largest known. There were few precipitation observations made in the area of heavy rainfall, but it was possible to estimate with reasonable accuracy the total storm rainfall from TVA flood history records of runoff which cover the major storm area. The resulting isohyetal map is shown in figure 30. Table 4 lists the maximum average depth of rainfall over various size areas for successive 24-hour increments.

TABLE 4.—*Storm of March 1-7, 1867.*

	Duration and maximum average depth of rainfall					
Area in square miles	Inches in 1 day	Inches in 2 days	Inches in 3 days	Inches in 4 days	Inches in 5 days	Inches in 7 days
10	8.1	10.3	11.9	14.6	15.9	16.0
100	7.6	9.9	11.4	13.9	15.1	15.3
200	7.4	9.7	11.1	13.6	14.7	15.0
500	7.1	9.4	10.7	13.2	14.3	14.6
1,000	6.8	9.0	10.3	12.9	13.9	14.2
2.000	6.6	8.5	10.0	12.6	13.5	13.8
5,000	6.1	7.8	9.4	12.0	12.9	13.2
10,000	5.7	7.2	8.9	11.3	12.2	12.5
20,000	5.1	6.4	8.1	10.3	11.1	11.4
50,000	4.0	5.1	6.6	8.5	9.2	9.5
64,000	3.7	4.7	6.1	7.9	8.6	8.9

Storms from 1875 to 1936

1875—The storm of February 23-25, 1875, produced the largest amount of rainfall of all storms of the convergence type which have occurred over the Basin during the period of record. The flood stages resulting from this storm on the Tennessee River at Chattanooga and points between Chattanooga and Knoxville have been exceeded by only one or two storms[1] of any type during the period of record. The circulation usually associated with a cyclonic disturbance did not play an important part in producing the rainfall in this storm until near the end of it. The airflow pattern in the storm was unusual in that the zone of greatest convergence and rainfall was not near the path of the center of lowest pressure but remained a considerable distance to the east and southeast of it through most of the storm.

1882—L. M. Pindell, Weather Bureau Observer in charge of the Chattanooga office in 1896, in *A Paper on the Tennessee River and Flood System,* gives data on a storm occurring in January 1882 which resulted in a stage of 40.4 feet at Chattanooga. Storm rainfall at Chattanooga and Knoxville totalled slightly over 10 inches. This storm was preceded by a snowfall at Chattanooga of about 18 inches.

1884—Mr. Pindell also reported data on the "freshet" of March 1884, resulting from total rainfall above Chattanooga varying from 2 to 6 inches. The total snowfall previous to the rise varied from about 2 inches at Charleston, Tennessee, to 20 inches at Leadvale, Tennessee. A crest stage of 42.9 feet occurred at Chattanooga.

1886—The storm of March 25-31, 1886, was a wave disturbance along a quasi-stationary front, followed by cyclogenesis, stagnation, and occlusion west of the Appalachian Mountains. The storm produced a crest stage of 52.2 feet at Chattanooga, the third highest stage of record at Chattanooga. A very large portion of the rain fell over the area which is now subject to control by only the main river dams.

Meteorologically, the storm represents the type which occurs in milder form over the southeastern United States as often as three or four times a year. The actual sequence of events during this storm is related in the following paragraphs.

During the night of March 25-26, a weak cold front advanced over the Tennessee Valley area and became quasi-stationary along a line just south of the area. Only light rains occurred in connection with the passage of this cold front. During the afternoon of March 26, a southward surge of the cold air over western and central Texas was balanced by a northward thrust of moist air over the central Gulf states and the development of a wave disturbance along the front over nothern Louisiana. This wave oscillation moved rapidly northeastward along the front during the next 24 hours. The lifting of the warm, moist air which resulted from this oscillation produced rain ranging from 1 to 2 inches from northern Mississippi and western Tennessee east-northeastward to southern and central Virginia during this period. Relatively cool and fair weather over the area following during the night of March 27 and morning of the 28th as the cold air following on the wake of the wave spread steadily southward over the South Atlantic states.

1. The crest stage on March 1, 1875, at Chattanooga was 53.8 feet. The unregulated flood of January-February 1957 would have produced a crest of 54 feet, which would have been the second highest stage of record at Chattanooga.

In the meantime, a greatly reinforced surge of very cold air over the Rocky Mountain and Western Plains states moving southward into New Mexico and the Texas Panhandle was again counterbalanced by another thrust of warm, moist, and unstable air over eastern Texas and Louisiana, causing another wave oscillation and cyclogenesis southwest of Tennessee. As the second wave deepened and moved slowly eastward in response to the surge of very cold air to the west, the balancing northward moving current of warm air was more and more confined by the block of cold air which had spread over the Atlantic Seaboard as far south as Georgia. Moderate to heavy rain continued over Tennessee, northern Alabama, and Georgia, and western North Carolina through the 29th, reaching greatest intensity over the area above Chattanooga on the 30th, as the cold air from the west converged upon the combined block of the Appalachian Mountains and the cold air lying along the Eastern Seaboard.

Finally, the continued eastward sweep of the cold air over the Gulf and South Atlantic States cut off the effective moist air supply and pushed the occluding system rapidly northward from northern Georgia on the evening of the 30th to central Ohio by the morning of the 31st, thus ending the storm.

1897—The storm period of March 3-19, 1897, is important historically in the Tennessee Valley area because it produced the greatest flood of record on the lower Tennessee River from the confluence with Elk River above Florence, Alabama, to the mouth. This was a sequence of storms occurring over a period of about two weeks with no one of the individual storms being important.

There were five distinct periods of intense rainfall over the Tennessee River Basin during the period from March 3 to 19, all of which were produced with intervals of 2 to 4 days between storm periods. The first four storm periods, namely, those of March 4-5, 8-9, 11-12, and 14 were of short duration (4 to 6 hours) high intensity rainfalls produced by type II storms. These rains occurred in varying width, warm sectors of storms passing to the northwest and north of the Basin and were produced by both prefrontal and cold frontal action. Intense convergence accompanied these storms as indicated by severity of thunderstorms, squalls, and hail. Rainfall amounts were limited by the speed of translation of these systems.

The fifth storm period of March 18-19 was also of type II except that the cyclogenesis again lying to the northwest of the region was of larger extent, deepened, and moved slowly northeastward. This period differed basically from the earlier periods only by the width and duration of moisture inflow, accounting for the greater accumulation of rainfall. The earlier rains were centered over the central Tennessee River while the last storm concentrated over the lower river, being centered in the vicinity of Florence, Alabama, as the earlier runoff was peaking, thus causing an accumulative effect. In the 17-day period, the total rainfall varied from an average of 14.6 inches over a 10-square mile area to 11.7 inches over 43,500 square miles.

1901—The storm of May 21-23, 1901, was one of the most severe known in the upper eastern part of the Tennessee Valley, particularly on the Watauga River and lower portions of the South Fork Holston and French Broad Rivers. It produced the highest stages of record on the South Fork Holston River at Kingsport and on the Watauga and Doe Rivers at Elizabethton, Tennessee. The storm was described in newspaper accounts as being "of unusual violence," and the mountain rivers rose very rapidly, causing much damage.

1902—The storm of March 25-29, 1902, produced one of the greatest known floods on several of the tributaries of the middle and lower Tennessee River including Richland Creek and the Duck River. This storm covered a large area with its center of maximum rainfall near Ripley, Mississippi. Precipitation data obtained from the storm studies of the Corps of Engineers show that slightly over 6 inches fell in 6 hours at the storm center with a total storm rainfall of almost 12 inches in 114 hours. An area-depth curve for this storm is included in figure 31 with others for storms of later years. The 1902 crest stages of the Duck River at Shelbyville, Tennessee; Elk River near Prospect, Tennessee; and Richland Creek at Pulaski, Tennessee, are the highest known. Rainfall totaling 5 to 10 inches fell over the Duck River Basin above Centerville during the four-day period, producing the second highest known flood peaks at that point and at Columbia. The flood on Richland Creek approached the maximum probable flood that might occur on a watershed of that size and location.

1916—The storm of July 13-17, 1916, was the second of two decadent hurricanes which brought heavy rainfall over the southeastern portion of the country during that month. The first storm moved inland over western Florida and Alabama, dissipating there during the period July 5-10, 1916. The heaviest rainfall associated with this earlier storm occurred over southern Alabama, but substantial amounts fell over the southeastern portion of the Tennessee River Basin, establishing ground conditions conducive to high runoff when the second storm arrived. The second storm moved inland as a well-defined hurricane near Charleston, South Carolina. No upper air soundings were available to determine the moisture charge and potential instability of the air which was flowing inland in connection with this disturbance. However, the air was undoubtedly highly charged with moisture and unstable, as evidenced by the fact that the orographic effect of the mountain slopes already was sufficient to release some of this moisture in the form of rain over western North Carolina. Furthermore, the surface dew points

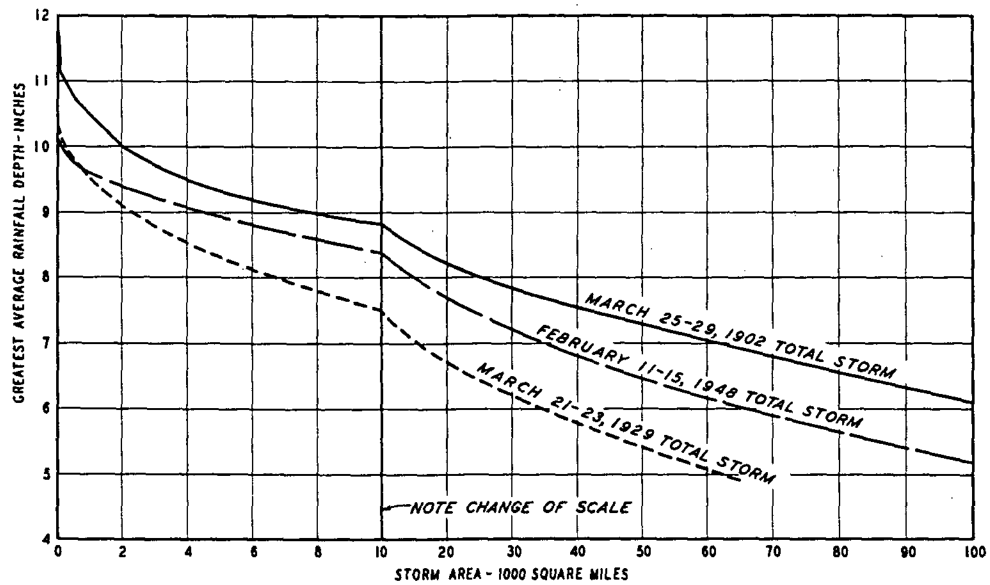


FIGURE 31.—*Area-depth curves for storms of March 1902, March 1929, and February 1948.*

were within two degrees of the maximum observed for the season, giving additional evidence of the high moisture charge present.

During the ensuing 24 hours, the storm moved steadily northwestward to western North Carolina. With the exception of the rain over the southeastern slopes of the mountains, the rain during the period was confined to the northeastern quadrant of the storm—most frequently the area of greatest rainfall activity in a tropical hurricane. By noon of July 15, the storm center had moved over western North Carolina. Although the central pressure of this storm had increased to such an extent that it could be identified only as a zone of moderately low pressure, the convergence associated with the low pressure area and the supply of moist unstable air remained strong. The heaviest rain occurred during the 18-hour period between noon on July 15 and 6:00 a.m. July 16. During this period, the center of low pressure moved slowly northward to southern Ohio where it dissipated and lost its identity.

A third hurricane moving over Haiti by the morning of July 16 sustained the flow of moist, unstable air and was probably instrumental in prolonging the duration of rain over the mountains of western North Carolina. The amount of rainfall over these mountains resulting from the mid-July hurricane is shown on the isohyetal map, figure 32. The centers of high rainfall reached 23.7 inches at Altapass, North Carolina, and 16.8 inches at Kingstree, South Carolina. The Altapass observer measured 22.22 inches between 2:00 p.m. July 15 and 2:00 p.m. July 16, the greatest amount in a 24-hour period ever recorded in the Tennessee River Basin. The area-depth curve for this storm is included in figure 33 with those for other large storms which affected the Basin. As a result of the intense heavy rainfall, the greatest flood occurred in western North Carolina since the first settlement and development of this region.

1917—The storm of March 1-5, 1917, was an important one in the northeastern part of the Valley. It produced the sixth ranking flood on the Clinch River at Clinton, Tennessee. Unusually heavy rains occurred during the first five days of the month over the entire Clinch River Basin, with heaviest amounts on the lower half of the Basin. The precipitation for seven stations in or near the Basin is as follows:

Station	*Total rainfall March 1-5, 1917, in inches*
Kingston, Tennessee	6.53
Clinton, Tennessee	6.69
Tazewell, Tennessee	6.10
Rogersville, Tennessee	5.98
Speers Ferry, Virginia	4.63
Elk Knob, Virginia	5.48
Burkes Garden, Virginia	3.86

This storm also produced a high ranking flood on the Tennessee River below the mouths of the Clinch and Hiwassee Rivers. At Chattanooga, the March 7, 1917, flood was the fourth highest of record, reaching a crest stage of 47.7 feet (fig. 34).

1929—The storm of March 21-23, 1929, ranks near the top of the list of important storms for the Tennessee River Basin because it set the upper limit of precipitation for durations of 12 hours over areas of 2,000 to 20,000 square miles. This storm is classified as a type II or convergence type. The

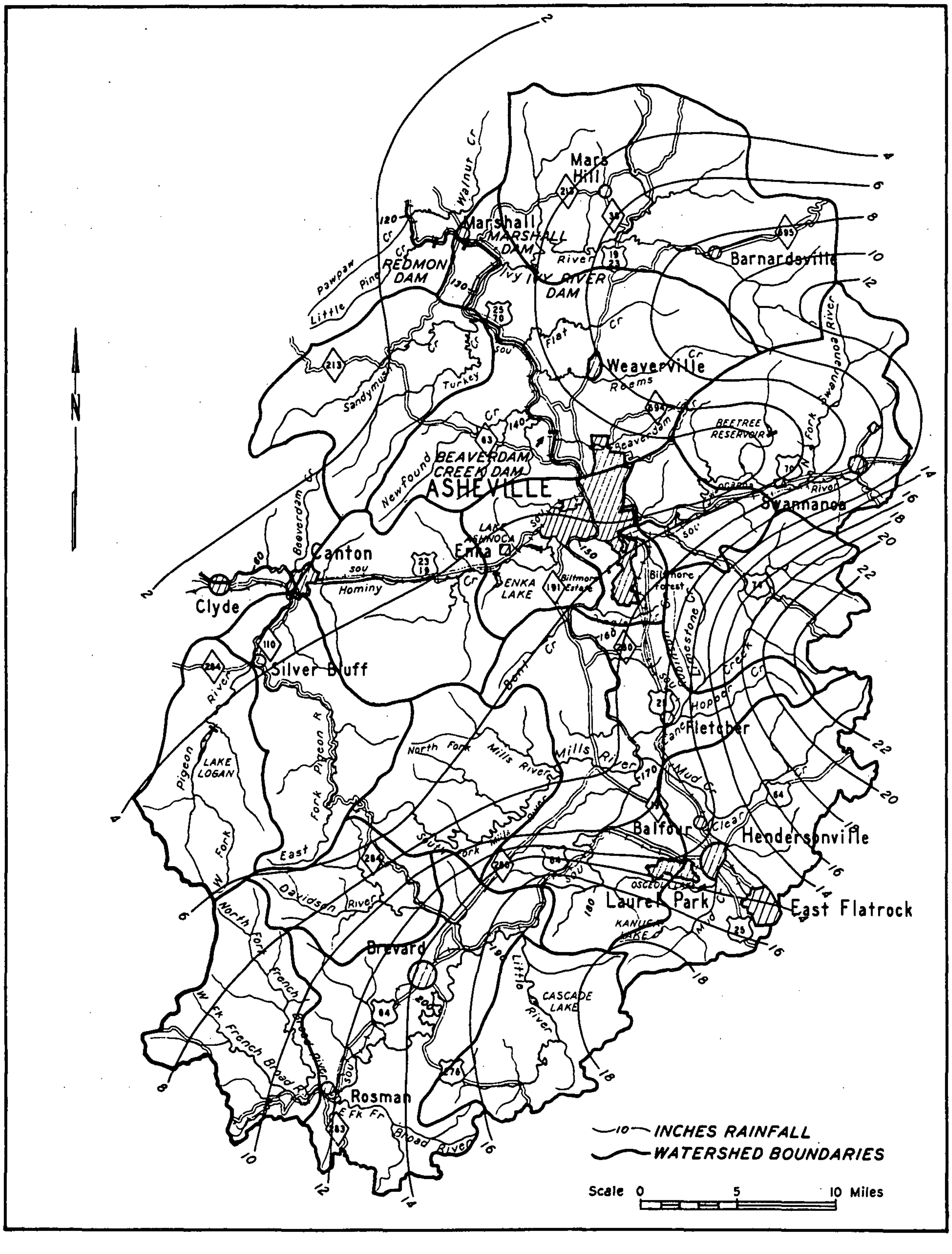


FIGURE 32.—*Rainfall over Western North Carolina mountains resulting from mid-July 1916 hurricane.*

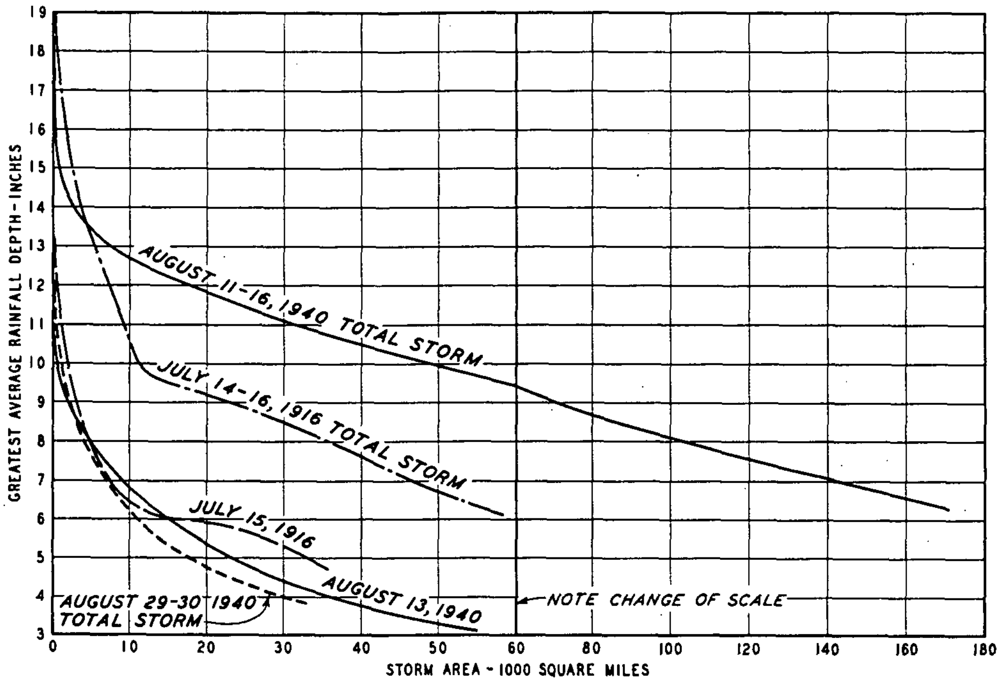


FIGURE 33.—*Area-depth curves for storms of August 1940 and July 1916.*

FIGURE 34.—*Chattanooga street scene during 1917 flood (Cline Photo Collection, Chattanooga).*

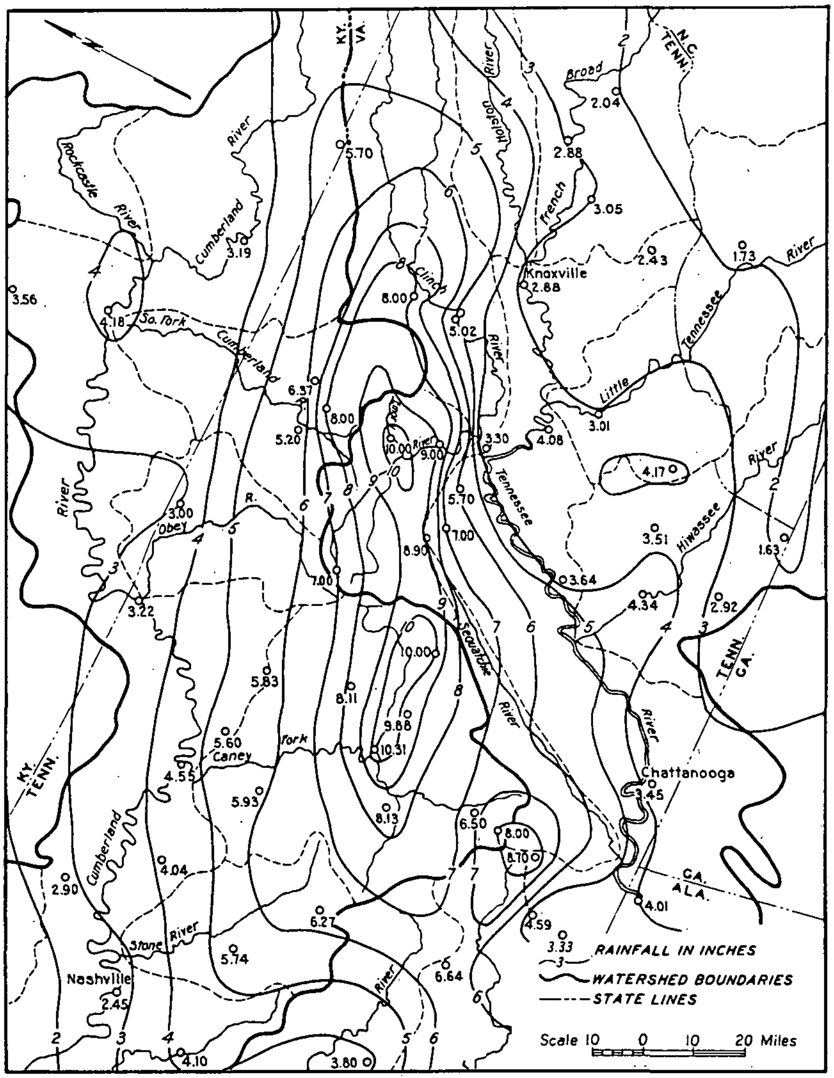


FIGURE 35.—*Rainfall over eastern Tennessee—storm of March 21-23, 1929.*

moisture charge of the inflowing air was very high as evidenced by the observed dew points at Birmingham, Alabama, a representative point of inflow, which very nearly approached the maximum for this season. Just preceding the 12-hour period of heaviest precipitation, a trough of low pressure extended from a center over southern Minnesota southward through Texas with a steep gradient from Texas eastward to Bermuda. During the 12-hour period of heaviest precipitation, the isobaric pattern showed a very rapid change from a straight-parallel, north-south orientation to a marked cyclonic curvature over the Gulf States, connoting considerable deceleration and convergent flow across the storm area. The average surface wind at Birmingham on the inflow side of the storm area was nearly double that observed at Knoxville on the outflow side.

The March 1929 storm produced the highest known flood on the Emory River and the second highest of record on the upper Duck River. Great destruction and loss of life occurred in the Emory River Basin, particularly in the vicinity of Harriman and Oakdale, Tennessee. Slightly over 9 inches of rain fell within 24 hours over the drainage area above Harriman, of which about 72 percent ran off. About 8 inches of the rain occurred in a 12-hour period. An isohyetal map of the total storm rainfall over eastern Tennessee is presented in figure 35 and the area-depth curve is included in figure 31.

1936—Heavy storms occurred on January 18 and 19, 1936, and on February 2 to 4, 1936, inclusive. These storms produced severe floods in the Hiwassee River Basin. The January storm delivered an average of about 4 inches over 2,300 square miles of the Hiwassee River watershed while the February storm averaged about 3½ inches. Although the precipitation during the February storm was less, the resulting flood stages at Murphy, North Carolina, and downstream were higher due partly to the presence of 2 to 10 inches of snow on the ground when the storm began and to completely frozen ground and other conditions conducive to high runoff. The highest recorded precipitation for the January storm was 5.8 inches and for the February storm 5.1 inches. Several stations registered 3 inches or more in 24 hours.

Storms since 1936

1940—The eastern section of the Tennessee River Basin was subjected to two major storms during August 1940 which produced floods exceeding the July 1916 crests on many headwater streams. The first storm occurred as a decadent West Indian hurricane and the second as a purely local meteorological disturbance. Each storm is described separately.

The mid-August 1940 storm was of tropical origin, having developed in the Atlantic Ocean east of the Bahamas sometime prior to August 8, 1940, when it first became noticeable on the weather map. It was of hurricane intensity when it moved inland over Georgia and South Carolina. The path of the storm is unusual for a West Indian hurricane, in that it moved inland a considerable distance and then recurved, describing the greater portion of a circle as its center passed approximately over Savannah and Atlanta, Georgia; Chattanooga, Tennessee; Bristol, Tennessee-Virginia; and Greensboro, North Carolina (fig. 36).

Since tropical storms usually drift in the direction of prevailing winds, the reason for this unusual trajectory is found in an analysis of upper air wind directions. In this case winds were predominantly east and east-southeast as far inland as Atlanta, due to a considerable westward displacement of the permanent Atlantic anticyclone. This resulted in a west-northwesterly movement of the storm as far as Atlanta. At this point the more southerly winds caused it to recurve and move in a northerly direction toward Chattanooga and Bristol. The final portion of the arc from Bristol to Greensboro was caused by the prevailing easterly winds still present near the coast.

The storm probably reached its maximum intensity with winds in excess of 75 miles per hour during August 11, when it moved inland near Savannah. Normally, this type of storm dissipates rapidly when moving inland due to the absence of sufficient moisture and the increased surface friction over land. In this case, the storm decreased in intensity near the surface, but maintained its intensity and high moisture content at higher levels.

The rainfall which accompanied the mid-August storm over the southeastern United States has already been shown earlier in this chapter in figure 27. The path of the rainfall parallels roughly the path of the storm center and forms approximately a large letter U with the base along the Blue Ridge Mountains of western North Carolina, one arm extending to Savannah, Georgia, and the other from the Virginia state line to the North Carolina coast line.

Centers of high rainfall reached 14 to 16 inches along the Blue Ridge in western North Carolina, 13 inches in Georgia and South Carolina, 15 inches in southern Virginia, and 18 inches on the coast of North Carolina. Central North Carolina received only 4 to 6 inches of rain although completely encircled by areas of much heavier rainfall.

Rainfall occurred principally during a 48-hour period from August 11 to 13 while the storm center traveled in an arc from southeastern Georgia to northeastern Tennessee. During this time the counterclockwise air movement about the center was approximately normal to the southern Appalachians, which is the most effective direction to produce lifting of the air. The vertical motion produced by the physiographic features caused precipitation increments in excess of those produced by the storm alone.

Rainfall intensities in this storm, although heavy, were not excessive, being generally less than 1 inch per hour.

The late August 1940 storm occurred during about a 24-hour period of August 29 and 30. Unlike the storm of mid-August, it did not originate as a well-defined storm center and it produced heavy rains only along the Southern Appalachian Mountains of eastern Tennessee and western North Carolina.

The storm occurred after a broad and relatively deep current of moist, tropical air had been flowing over this region for a number of days, producing scattered showers and thunderstorms. During the night of August 29-30, a mass of cool polar air moved southward along the Atlantic coast. This resulted in a general steepening of the isentropic surfaces over this region, with the greatest slope in a northeast-southwest axis. A strong southerly current carried tropical air up these rather steep isentropic surfaces. This resulted in releasing the available energy of the unstable tropical current of air, which in turn produced heavy shower activity. The physiographic features of the area undoubtedly aided in initiating the release of shower activity.

The rainfall in the late August storm over the southern Appalachian region is shown on the isohyetal map, figure 37. Rainfall amounts up to 13 inches occurred in a relatively narrow band along the Blue Ridge in western North Carolina. Total rainfall decreased rapidly in all directions from this mountain ridge, so that at about 50 miles distant only about 2 inches was recorded.

Rainfall intensities at the stations receiving heavy rainfall were generally higher than in the mid-August storm. Maximum intensities ranged between 1 and 2

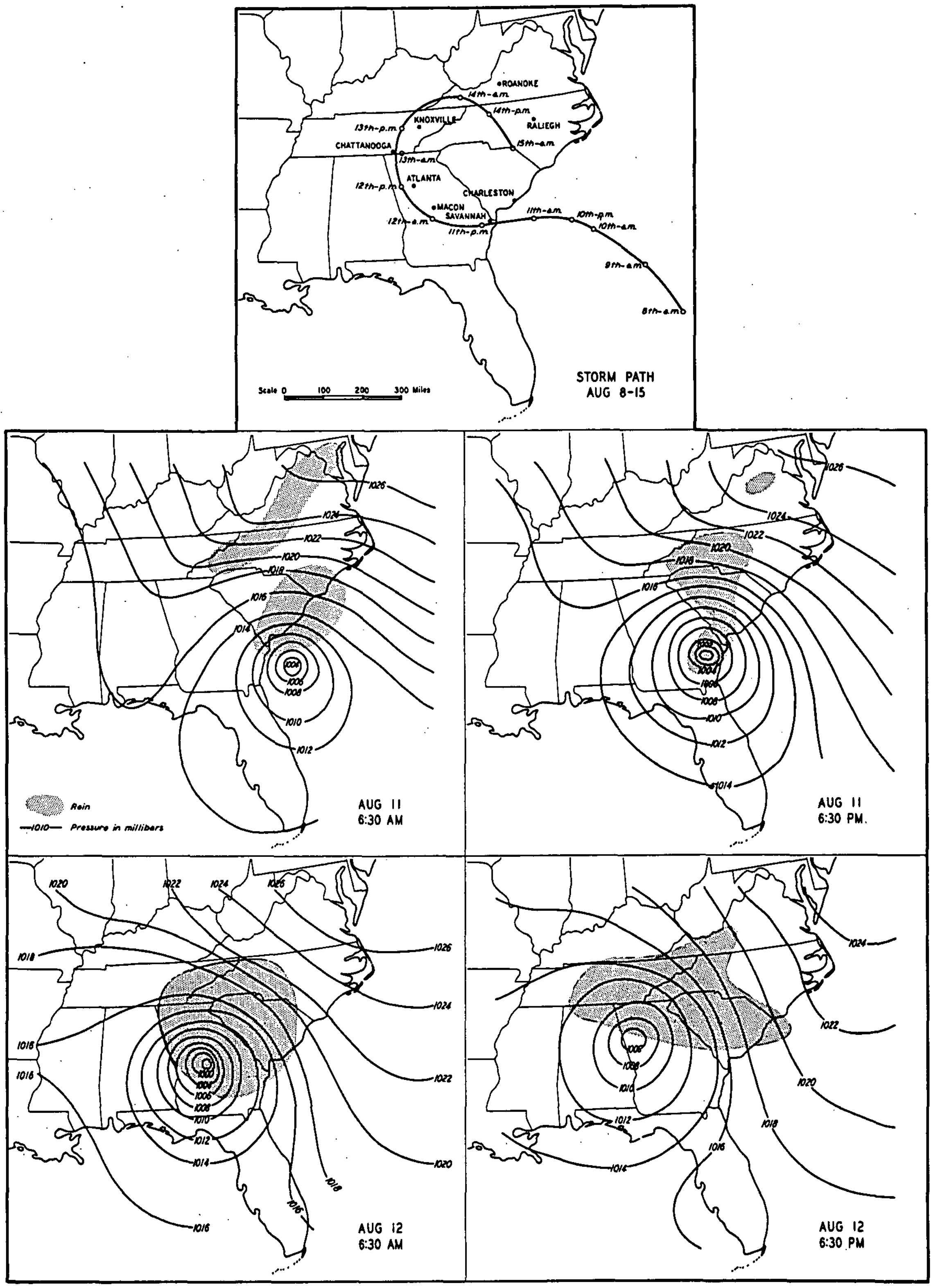
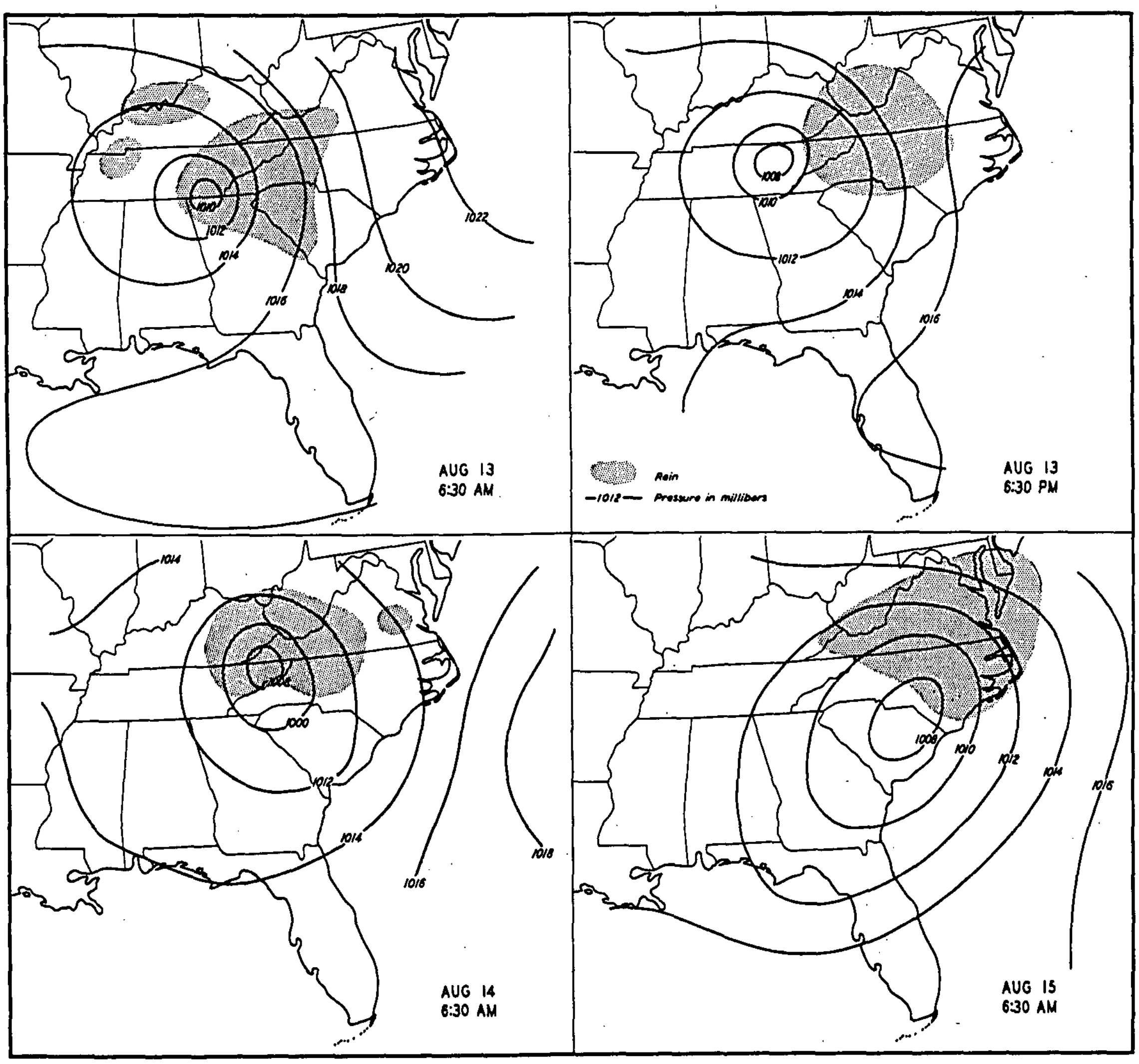


FIGURE 36.—*Weather maps-*



m of mid-August 1940.

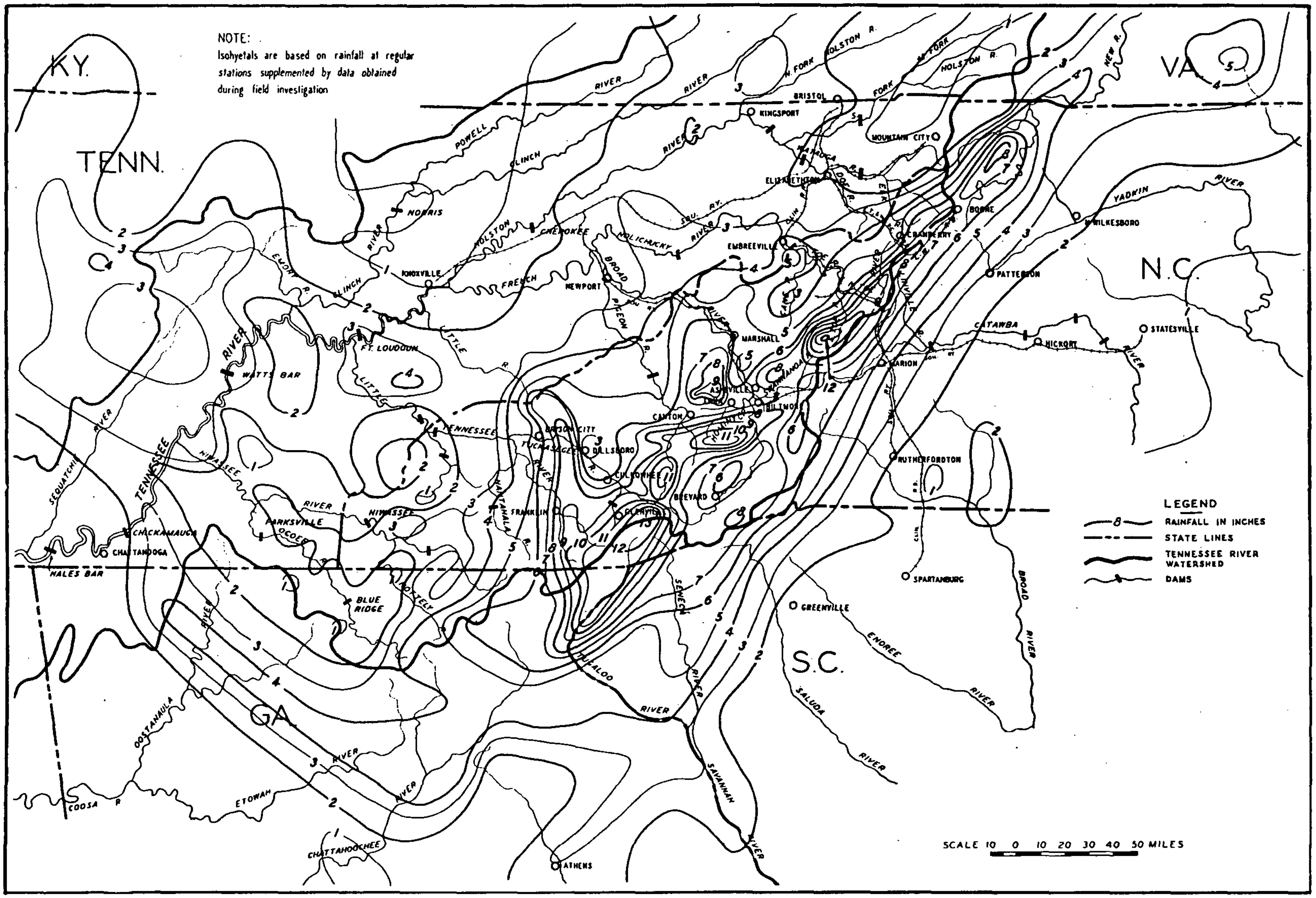


Figure 37.—*Rainfall August 29-30—storm of late August 1940.*

inches per hour except at Mount Pisgah where they exceeded 2 inches. Figure 33 shows area-depth curves for the total storm rainfall periods of August 11-16 and August 29-30 and also for August 13 which was the day of maximum rainfall for the whole storm area.

In many of the headwater streams, the height of the August 1940 floods exceeded any known past floods. The volume of flood water from the French Broad, Little Tennessee, and Hiwassee Rivers raised the Tennessee River to flood stages from Knoxville to the head of Chickamauga Reservoir during both floods.

1948—The storm of February 11-15, 1948, was a type I storm or wave disturbance along a quasi-stationary front. The storm over the Basin was a part of a general storm which covered an area 650 miles long, extending from northeastern Louisiana to northeastern Kentucky in a band 250 miles wide. The greatest amounts of rainfall occurred over the west-central and southwestern part of the Tennessee Valley. Within the storm center, as much as 10.4 inches of rain fell in a period of about 48 hours. The area covered by depths greater than 9 inches was 4,500 square miles and by depths greater than 6 inches 65,000 square miles. The storm rainfall came on watersheds nearly saturated from rainfall and snowfall which had occurred during the 30 days prior to the storm. Hydrologic factors were conducive to a maximum rate of runoff. The resulting floods on the Duck River from Columbia downstream and on the Buffalo River broke all known records.

The isohyetal map of the total storm rainfall is presented in figure 38. An area-depth curve for this storm is included in figure 31. The bulk of the rain fell in a 48-hour period from the evening of February 11 to the evening of February 13. In most localities, the rainfall was not continuous, occurring during two or three periods of moderate intensity rainfall with light or no rainfall between bursts. Despite the heavy storm rainfall, relatively few stations experienced hourly intensities in excess of 1 inch per hour.

1950—Precipitation in the Tennessee Valley during the three months of January through March 1950 was the greatest for any like period since 1891 and the second greatest in 61 years of available record. In the portion of the Valley below Chattanooga, the precipitation in the first three months of 1950 exceeded by 3 inches the previous 61-year record. Although no exceptional floods resulted on tributary streams of the Tennessee River from these 1950 storms, the accumulation of heavy inflow to the lower river threatened to aggravate serious flood conditions already prevailing on the Ohio River at Cairo, Illinois.

There were three storm periods outstanding in the January-March 1950 siege of heavy rainfall in the Tennessee Valley. All of these storm periods were associated with wave disturbances formed along a quasi-stationary front. The first, on January 4-6, was confined almost entirely to the western half of the Valley. Total precipitation exceeded 8 inches in parts of northern Alabama. The second storm, from January 29 to February 2, produced heavy rainfall across the northern half of the Valley and in the upper Caney Fork Basin. A maximum amount of 8.4 inches was recorded in the Clinch River area below Norris Dam and over 7 inches fell on the upper Duck River watershed. The third storm, on March 11 to 13, was heaviest in the southern half of the Tennesse River Basin. Total precipitation of 5 to 6 inches fell in northern Alabama and in the Hiwassee and Little Tennessee River headwaters.

Numerous minor storms occurred during the three-month period. The three largest of these were on January 18-19, February 6-9, and February 12-13. A comprehensive coverage of the meteorological conditions for each of the significant periods of precipitation during the three-month period is contained in the TVA report, "Tennessee River Basin Floods of January-March 1950."

1955—A heavy 24-hour storm on March 20-21, 1955, produced major floods in the western half of the Valley. The distribution of rainfall was very similar to that of February 1948 with amounts of 3 to 11 inches falling over an area 650 miles long and up to 170 miles wide extending from northern Louisiana to south-central Kentucky. Within the Valley the maximum amount recorded was 10.12 inches at Iron City, Tennessee. The resulting floods equalled or exceeded previous records on Cypress Creek near Florence, Alabama, and Big Rock Creek at Lewisburg, Tennessee. The flood was the highest since 1902 on Richland Creek, the lower Elk River, and Shoal Creek. Only the floods of 1948 and 1902 exceeded the 1955 crest on the Duck River at Columbia and Centerville.

The disturbance which brought this heavy rainfall developed in the southwest as a secondary disturbance from a parent low located over North Dakota on March 18. It continued to grow and move southeastward as the circulation aloft brought cold air southward. The flow of moist air from the Gulf of Mexico was intensified, sharpening the thermal contrast. During the late afternoon and night of March 20, this moist air was subjected to large-scale vertical lifting over Arkansas and western Tennessee as the new low-pressure system and trough aloft continued to move eastward. With ample moisture available, heavy rains fell over the Tennessee Valley during the 36 hours ending at 6:30 a.m. on March 22. The rapid movement of this storm over the Valley was somewhat unusual and this limited the total amount which might have occurred.

Figure 39 shows precipitation over the southeastern United States during the passage of the storm.

1957—During the first two weeks of January 1957, moderate to locally heavy precipitation oc-

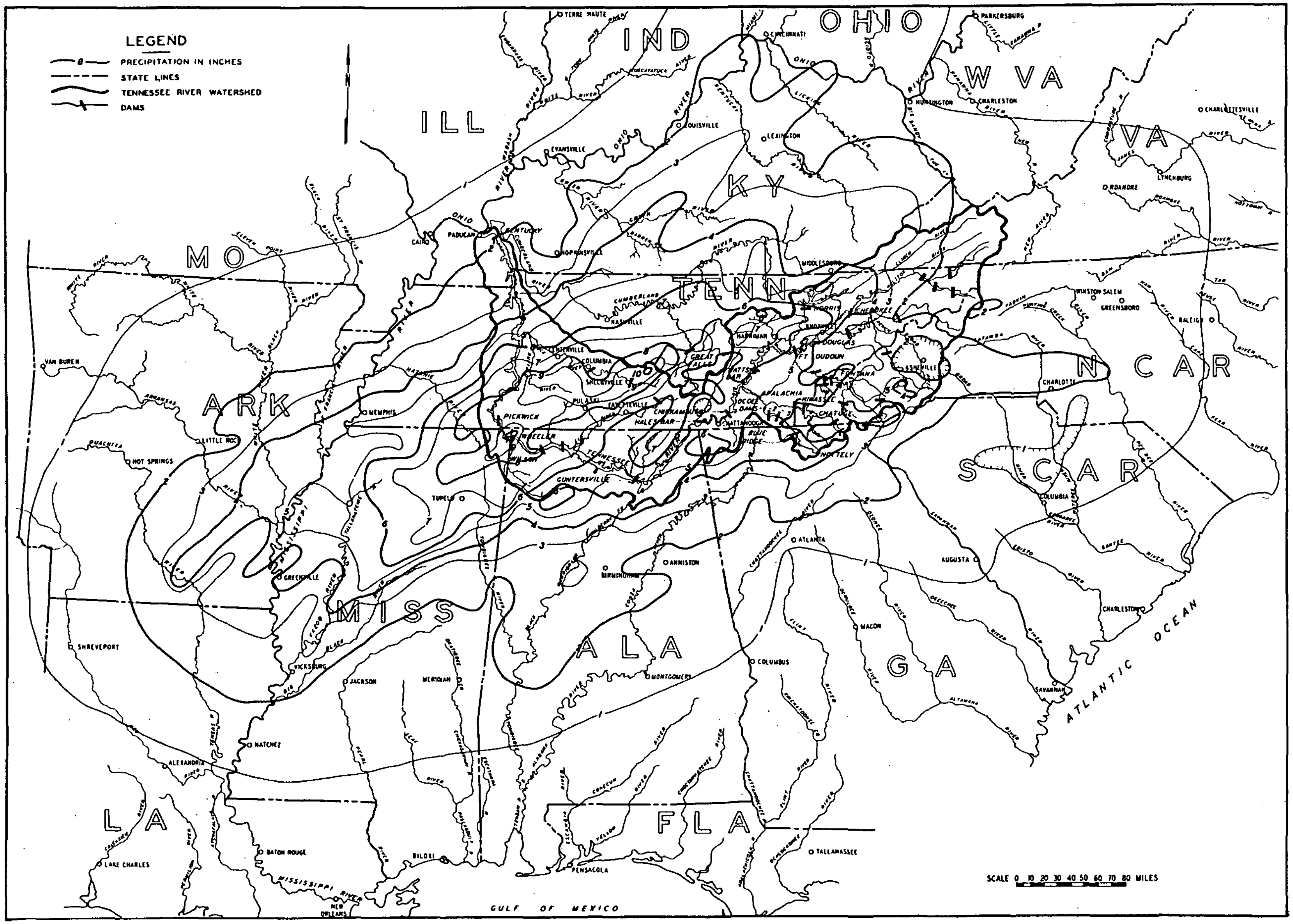


FIGURE 38.—*Rainfall February 11-15—storm of February 1948.*

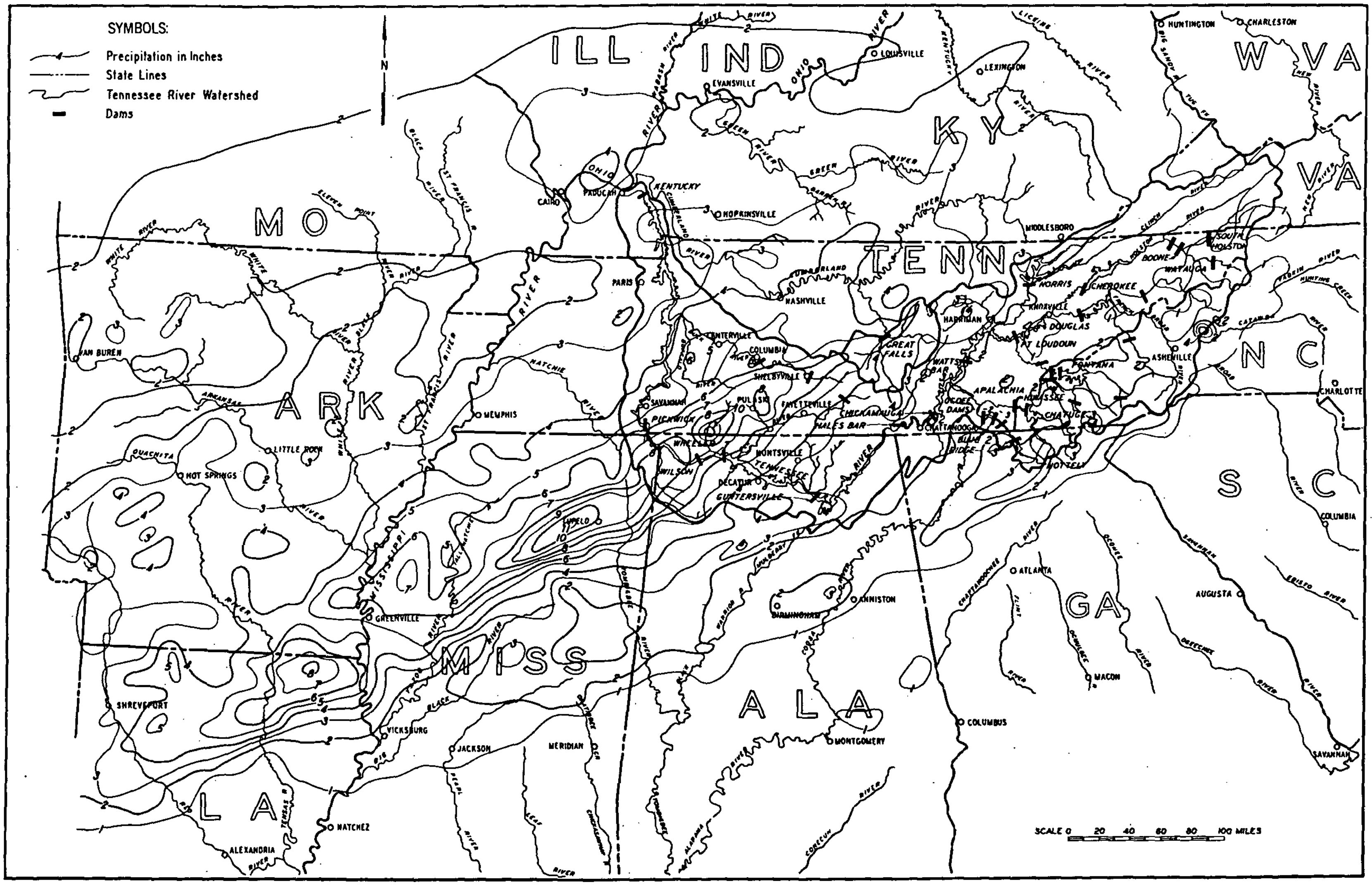


FIGURE 39.—*Rainfall over southeastern United States—storm of March 20-21, 1955.*

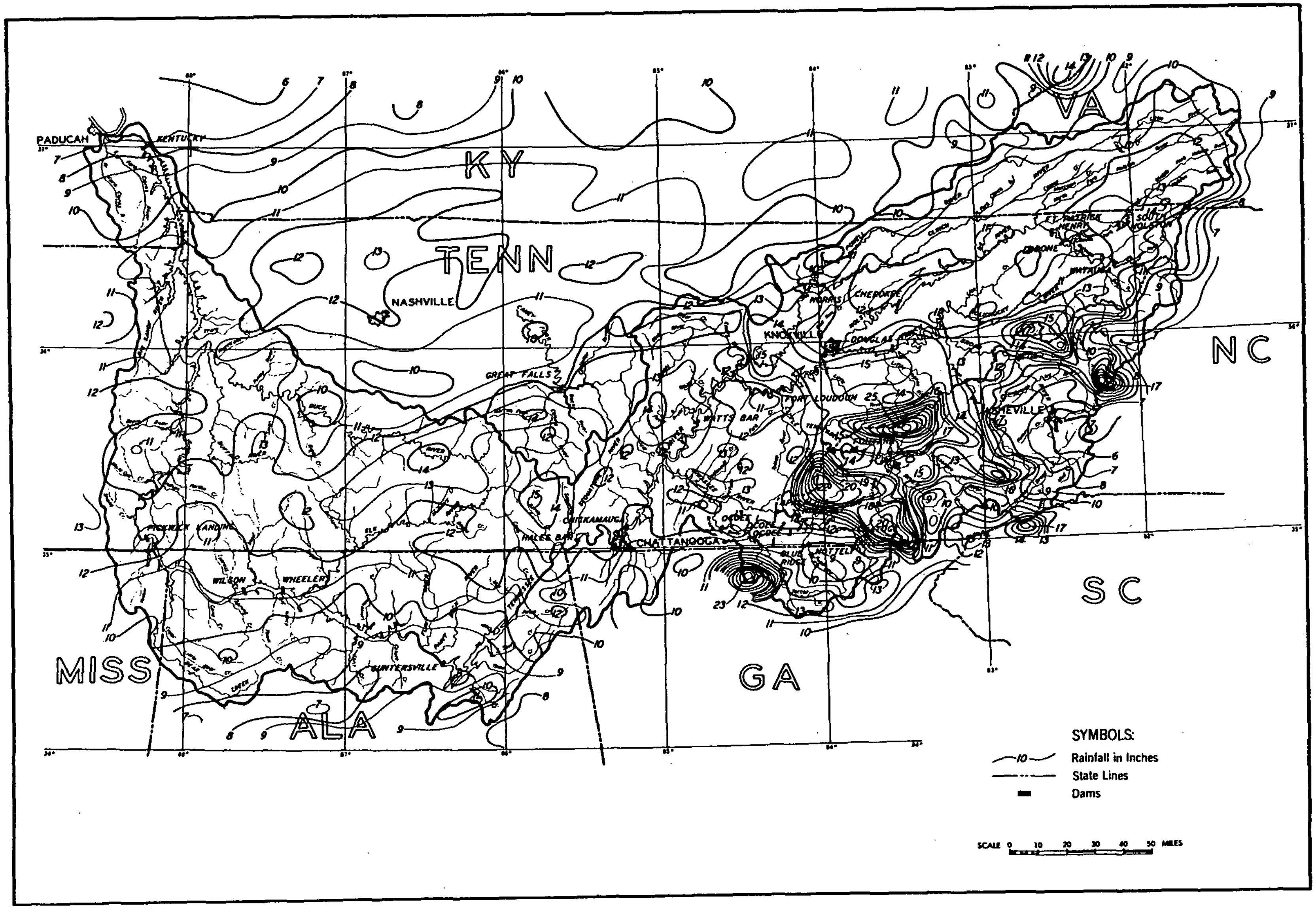


FIGURE 40.—*Rainfall over Tennessee River Basin—storm of January 21 - February 10, 1957.*

curred over the Tennessee Valley area at intervals of three to five days, marking the beginning of a regime which dominated the weather during the rest of January and early February. Intermittent moderate rains occurred over the eastern portion of the Valley during the third week of January. In the fourth week, cool air surged southward over most of the western two-thirds of the nation coupled with the intensification of a northeastward flow of warm, moist air from the Gulf of Mexico over the southeastern part of the country. The boundary zone between the cold and warm air remained near or over the Tennessee Valley area with only one short interruption. Numerous daily wave disturbances moved eastward through this boundary zone, effectively releasing the moisture in the warm air. During the last four days of January and first day of February there was a concentration of excessive rainfall over nearly the same path on successive days, causing serious flood conditions in the Tennessee Valley. This would have produced the second highest flood of record at Chattanooga, 54 feet, had it not been regulated.

Figure 40 is an isohyetal map of the total storm period of January 21-February 10, 1957, in the Tennessee Valley Basin, showing the distribution of record or near record breaking rainfall that occurred. The rainfall ranged from a maximum of 25.29 inches at the recording rain gage on Clingmans Dome, North Carolina, in the Great Smoky Mountains National Park to a minimum of 5.2 inches at the Asheville-Hendersonville airport station, about 60 miles east-southeast of Clingmans Dome. This wide variation in rainfall in short distances is characteristic of the mountainous southeastern section of the Tennessee River Basin. The rainfall throughout the remainder of the Basin was more uniform, varying from about 10 to 14 inches. The average rainfall for the 21-day storm period, January 21 to February 10, 1957, in the whole Basin was 11.7 inches, in the area above Chattanooga 12.2 inches, and in the area below Chattanooga 11.2 inches.

The storm rainfall occurred in three periods. The first period was from January 21 to 23 when the rainfall caused medium high stages on some streams and increased soil moisture to near saturation, thus making conditions ideal for heavy runoff from the succeeding periods of rain. From January 24 to 26 there was an interval of light rainfall. The next period, from January 27 to February 2, was the heaviest portion of the total storm. Rainfall in this period averaged about 7.3 inches or 62 percent of the total. Tributary streams reached their crest stages during this period, and the Tennessee River began its rise to near record flood stages. Light to moderate rainfall followed in the last period, February 3-10, adding large volumes of runoff to receding streams in the middle and upper portions of the Basin.

INTENSE STORMS OVER SMALL AREAS

Intense rainfall such as occurs during thunderstorms is seldom of a magnitude that affects seriously the large streams of the Tennessee River Basin. However, these storms may cause sudden and destructive floods on smaller watersheds. Characteristically, these storms cover relatively small areas with very intense rainfall which decreases rapidly and radially from a high point at the center. Because of the limited extent of the storms, the spacing of precipitation gages, even in what is normally considered a dense precipitation station network, is seldom close enough to define adequately the pattern of rainfall. As these storms are very important in studies of flood situations in communities on small tributaries in the Tennessee Valley region, TVA hydraulic engineers have made numerous field investigations immediately after severe local storms and floods in the Tennessee Valley and vicinity to ascertain accurately the amount, intensity, and areal extent of rainfall associated with the high runoff. Supplemental rainfall catches as well as runoff and damage data are obtained during these investigations. Information on the duration of the intense rainfall as well as the amount is secured from the local residents.

TABLE 5.—*Selected list of intense storms over small areas.*

				Extent of heaviest rain[1]		
No.	Date	Location	Duration, hours	Approximate area, square miles	Minimum amount in area, inches	Approximate rainfall at center, inches
1	June 13, 1924	Carter County, Tenn.	3½	50	5	15
2	August 2-3, 1939	Near Lebanon, Tenn.	16	600	5	14
3	June 21, 1956	Near Manchester, Ky	3	18	5	12
4	June 13-14, 1947	Near Mt. Airy, N. C.	30	1,600	6	11
5	June 13, 1952	Near McMinnville, Tenn.	3	420	5	10½
6	August 8-9, 1954	Near Dunlap, Tenn.	4	390	6	10
7	June 18, 1939	Near Lewisburg, Tenn.	3	400	3	9
8	June 28, 1947	Near Greeneville, Tenn.	3½	750	4	7½
9	July 28, 1947	Near Cosby, Tenn.	3	50	2	6½
10	July 16, 1949	Near Morristown, Tenn.	2	15	3	4½

1. The area shown is that receiving rainfall greater than the minimum amount listed.

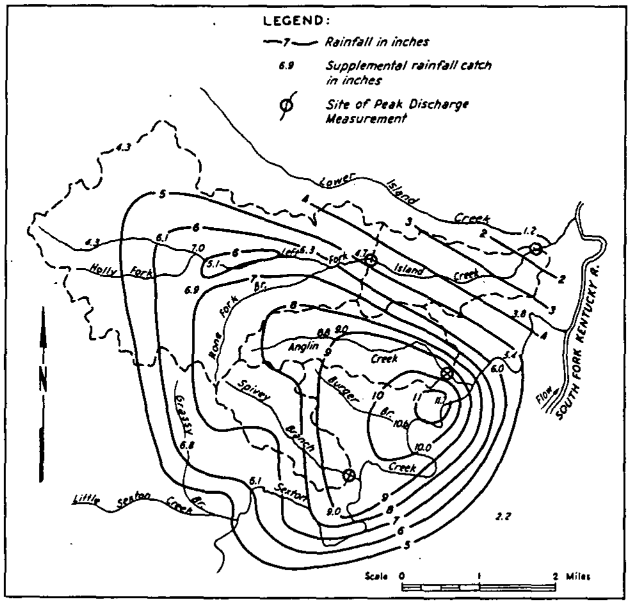


FIGURE 41.—*Rainfall from storm of June 21, 1956, near Manchester, Kentucky.*

These supplemental catches yield a much more complete picture of the rainfall distribution than is obtainable from existing precipitation records, thus helping to explain the resulting observed high peak rates and volumes of runoff. From these investigations, it has been possible to construct isohyetal maps of many severe storms over small drainage areas. A typical example of the coverage obtained from supplemental rainfall catches is shown by the isohyetal map (fig. 41) of the June 21, 1956, storm near Manchester, Kentucky.

Additional information on locally severe storms has been obtained from other available sources such as the Corps of Engineers, U. S. Geological Survey, and State of Tennessee Division of Geology. These storm data are utilized in studies of local flood situations for many communities, transposing them, with necessary adjustments for topography and geography, to ascertain maximum floods which could be reasonably expected to occur in those areas. A few of the many locally intense storms investigated have been selected and listed in table 5 and located by their approximate centers of high rainfall on the map shown in figure 42. Area-depth curves for the ten selected storms are presented in figure 43.

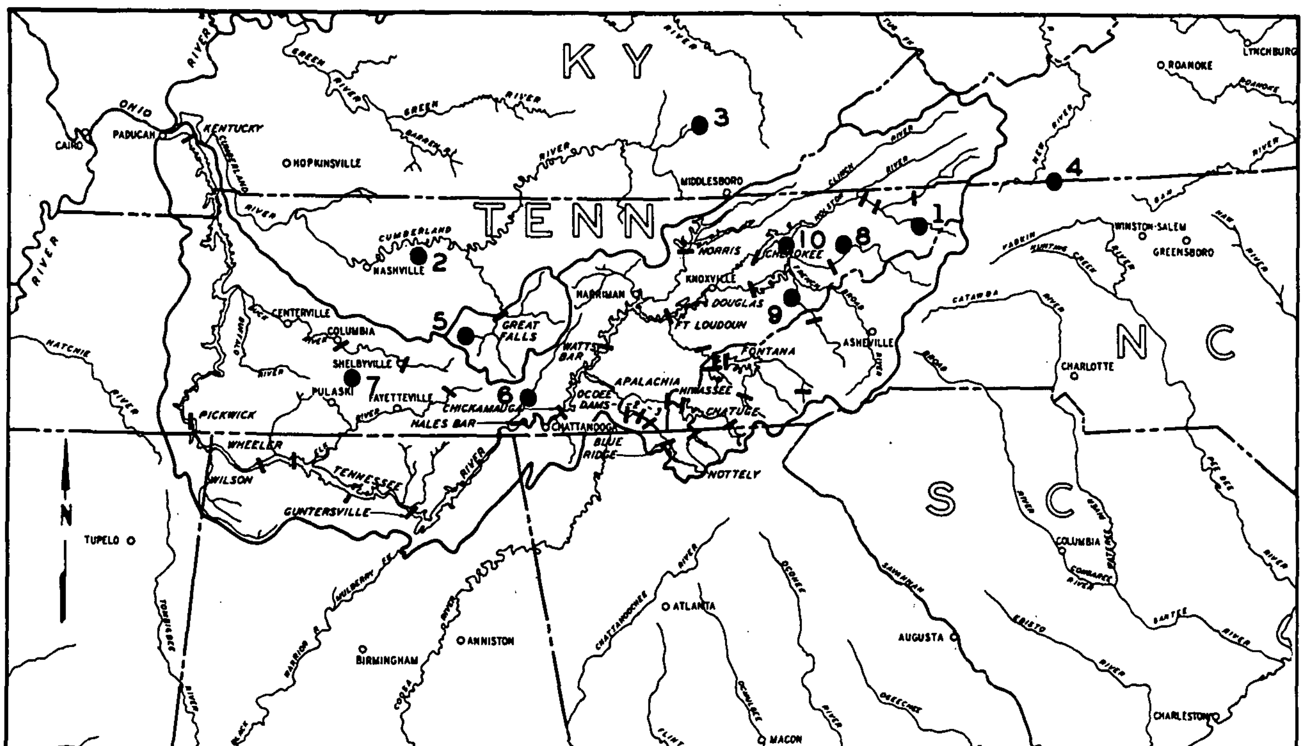


FIGURE 42.—*Locations of the intense storms over small areas listed in table 5.*

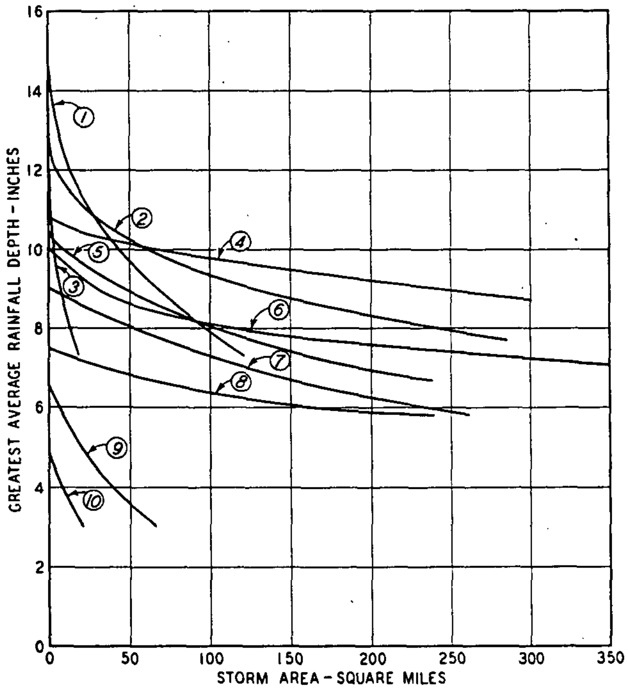


FIGURE 43.—*Area-depth curves—selected intense storms over small areas. (Numbers in circles refer to storms identified in table 5 and located in figure 42.)*

MAXIMUM STORMS

Use of observed storm data

Essentially, the determination of the flood that must be controlled or guarded against in any area is a meteorological matter. Therefore, consideration must be given to the meteorological potential of the area in order that adequate thought be given to that aspect, along with the economics, when deciding upon a design value. In evaluating the flood-producing meteorological potential over a given area, it is requisite that records of major storms and floods in the area be obtained. From these records the maximum storm which it is possible for a given area to experience can be determined, and this section discusses the methods used in determining this maximum.

The accumulation and analyses of maximum storm rainfall data have as a primary objective the estimation of great floods for engineering design purposes, and the principal methods used by TVA to do this are described in chapter 5.

Transposition of storms

The application of a storm occurrence over one area to another area within the same region of meteorological homogeneity is referred to as storm transposition in hydrometeorology and is accepted as a basic concept. Such transposition requires determination of whether a given storm in one area could have occurred in the other area.

Meteorological conditions—A region of meteorological homogeneity is one in which every portion can experience a storm event with the same storm mechanism and total inflow wind movement, but not necessarily with the same moisture charge or same frequency. Thunderstorms, being convective precipitation, can occur practically any place, so transposition is permissible without reservation of distance. On the other hand, decadent hurricanes from outside the area of homogeneity are not transposable because of the radical structural transformation which these storms undergo as they move farther from their source region and as they travel over land. Other storm types require careful study for permissibility of transposition. The area of meteorological homogeneity for the Tennessee Valley region studies has been defined as approximately 600 miles from north to south and 900 miles from east to west and it includes the states of Alabama, Arkansas, Georgia, Kentucky, Mississippi, North Carolina, South Carolina, Tennessee, and the Great Valley section of Virginia.

A second consideration in transposition should be in regard to shape and orientation of the rainfall pattern as defined by isohyetal lines. The elongated rainfall patterns which are associated with quasi-stationary frontal rains should not be changed in shape or greatly reoriented. The problem resolves itself into one of making decisions based upon the meteorological characteristics of both the area and the storm.

Time-area-depth relation—One of the most necessary and useful "tools" in storm transposition is the relation between the areal distribution of storm precipitation and its time distribution. The relationship is usually made available in tabular form or as a family of curves. The area-depth data are obtained by planimetering the total storm isohyetal map. The duration analysis is derived from mass curves which are graphs of accumulated storm rainfall versus time. From the isohyetal map and mass curve, maximum observed area-depth curves for various durations of the storm are developed. These values are used subsequently as observed rainfall to be transposed and adjusted for maximum factors to produce the maximum possible precipitation.

The total storm isohyetal map is planimetered, proceeding outward from the high rainfall center, to compute the average rainfall over increasing areas. The results of this computation form the area-depth curve for the total storm duration. For durations less than the total, values of area-depth are computed by averaging the mass curves of rainfall for individual stations. A total storm isohyetal map for the storm of February 1-5, 1936, is presented in figure 44 and the mass curves for individual stations in the storm area are presented in figure 45 along with the tabulation of maximum observed rainfall depths over different areas for various durations.

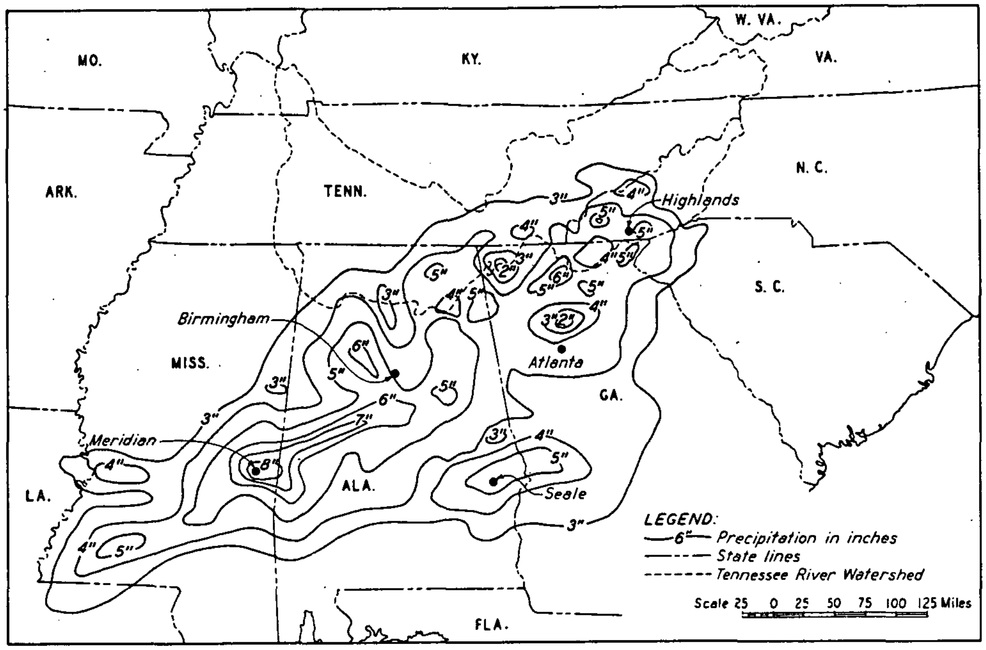


FIGURE 44.—*Rainfall—storm of February 1-5, 1936.*

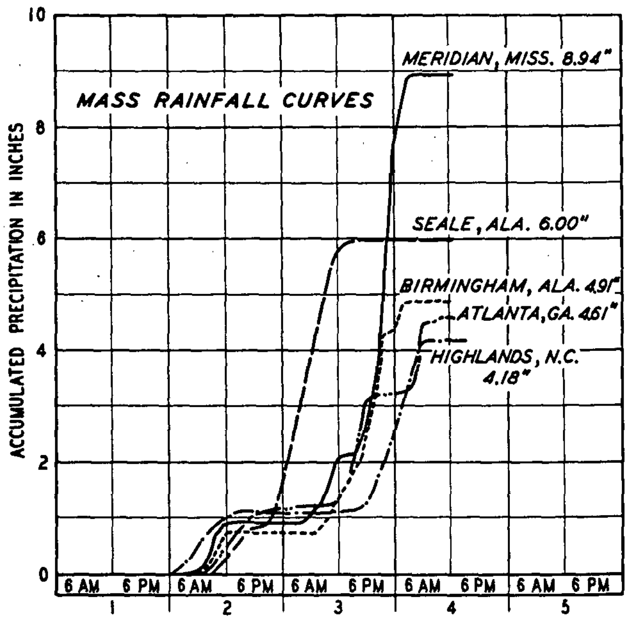
Maximum possible storm precipitation

The term "maximum possible precipitation" for a specific area and duration is defined as the depth of precipitation which can fall in the area but will not be exceeded under known meteorological circumstances. Because the natural laws limiting precipitation rates are not fully known, computations of such depths must be regarded as "best estimates." Like any estimate, there is implied a range of tolerance which depends upon available technical knowledge, deficiencies of data, and extent of analysis. The values are regarded as maximum possible, because they are derived, within the limits of existing knowledge and data, from the most effective combination of factors controlling amount of rainfall.

The maximum possible precipitation is computed by the adjustment upward of amounts of observed storm precipitation to the precipitable-water content appropriate to the maximum observed dew points in the area of study, with due regard to the influence of topographic barriers upon the moisture content of inflowing air. Increasing observed storm wind velocity to maximum value is generally not regarded as a sound adjustment. Although the velocity of inflow winds in a particular storm under study may be lower than the maximum, increasing the velocity would be unsound meteorologically if other factors were left unchanged. Adequate wind data are not generally available for most storms so that maximizing storm wind velocity becomes impracticable.

Starting with the premise that the dynamic characteristics of the storm will be unchanged in maximizing the precipitation, the principal factor affecting the amount of rainfall is the moisture charge. The moisture charge of the storm is indicated by representative surface dew points in the warm sector. Dew point is the temperature to which water vapor must be cooled to produce saturation. The surface dew point is a convenient reference to identify the potential moisture charge of a saturated column. It must be emphasized, however, that the surface dew point is representative only when the column of air is saturated and the vertical temperature distribution is the same as that described by a rising parcel of saturated air. This limiting condition is found in flood-producing rains, since large scale lifting is always associated with such storms. In the maximum storm studies conducted by TVA, a determination was made of the maximum possible dew points for durations up to 84 hours at a sufficient number of stations in the southeastern United States to define the seasonal trend and the geographical variations. This involved a search of the period of record of about 50 years at each station to determine the maximum observed dew point for each month and for varying durations. Considerable care was exercised in selecting representative data, and the results were correlated between stations. The maximum values determined in the study were used in adjusting observed storms to their physical upper limits.

An additional adjustment must be made in the moisture charge to account for the effect of the orographic barrier to inflowing air. Some of this moisture will be precipitated before the air can enter the area blocked by this barrier and, therefore, must



DURATION AND MAXIMUM AVERAGE DEPTH OF RAINFALL						
AREA IN SQ MILES	INCHES IN 6 HOURS	INCHES IN 12 HOURS	INCHES IN 18 HOURS	INCHES IN 24 HOURS	INCHES IN 48 HOURS	INCHES IN 90 HOURS
10	5.5	6.8	7.5	8.0	8.9	8.9
100	5.3	6.5	7.3	7.8	8.6	8.6
200	5.2	6.4	7.2	7.7	8.5	8.5
500	4.7	6.0	6.8	7.3	8.3	8.3
1,000	3.3	5.2	6.4	6.8	7.9	8.1
2,000	2.8	4.4	5.6	6.1	7.4	7.7
5,000	2.5	3.9	5.0	5.5	6.6	6.9
10,000	2.4	3.6	4.6	5.2	6.1	6.4
20,000	2.2	3.2	4.2	4.7	5.6	5.8
50,000	1.7	2.5	3.3	3.9	4.7	5.0
100,000	1.0	1.6	2.4	3.0	3.9	4.2

FIGURE 45.—*Mass curves and maximum depths of rainfall—storm of February 1-5, 1936.*

be subtracted from the effective precipitable water. The family of curves shown in figure 46 gives the effective precipitable water remaining in a column after it has undergone lifts ranging up to 10,000 feet. For example, from figure 46, air having a surface dew point of 80 degrees contains about 2.3 inches of effective precipitable water at sea level. If this column were lifted to 10,000 feet, only 0.9 inch would remain in the column. This chart is used to determine the effective precipitable water corresponding to maximum dew points in adjusting storms to their physical upper limits and transposing them to other areas.

The inflow barrier is defined as the average height of the highest continuous land barrier 5 miles or more to the windward side of the storm area. The distance of five miles is used as an average value to allow for the forward motion of precipitation as it falls earthward. At a velocity of 30 miles per hour, which is the order of magnitude of the storm winds experienced, rain travels approximately 5 miles forward in the time from formation until it hits the earth's surface.

Maximizing actual storms

One of the principal purposes of the maximum observed storm studies is to adjust and transpose various storms to maximize the amounts of rainfall over specific areas for study. The adjustment factors which have been used in TVA studies include the increase of observed surface dew point to the maximum for the storm date and location as well as correction for inflow barrier.

Figure 47 illustrates the method used. In this figure pertinent data used to effect the adjustment and transposition to the maximum over the Upper French Broad River Basin are given for the storm of July 13-17, 1916, on a copy of the form used in the maximum storm studies. The July 1916 storm was the result of a decadent hurricane and is described elsewhere in this chapter. Comparison of the dew points which prevailed during this storm with the maximum shows that the transposition and ad-

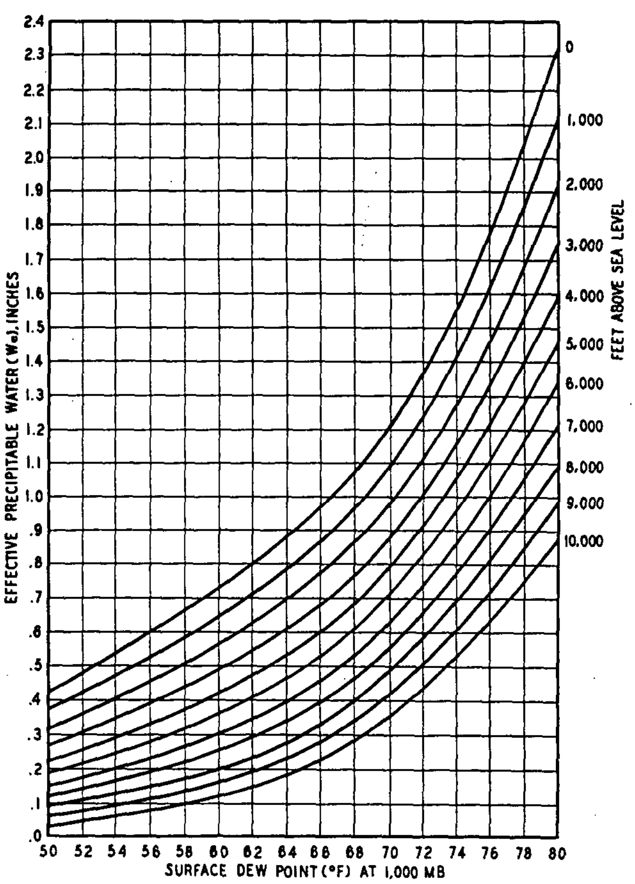


FIGURE 46.—*Effective precipitable water as a function of reduced dew point and lift of a saturated column of air.*

TVA 3414 (WCP-2-46)
Tennessee Valley Authority

Hydraulic Data Division

Observed and Maximum Factors of
Storm Adjustment and Transposition

Storm of July 13-17, 1916 Adjusted to Storm Date

Area Of Observed Storm	Area Of Transposition
Location Western North Carolina	Basin French Broad above Asheville
Inflow Direction SE-310	Inflow Direction SSE
Inflow Barrier H= 1600 Ft.	Inflow Barrier H'= 2000 Ft.
Dew Point Station Charlotte, N. C.	Maximum Dew Point Location From Inflow
Distance to Barrier 70 Miles	Barrier 50 Miles NNE of Augusta, Ga.

(1) T Hours	(2) DP(obs) Degrees	(3) DP(max) Degrees	(4) We(obs) Inches H= 1600	(5) We(max) Inches H'= 2000			(6) R	(7) D(obs) Inches	(8) D(max) Inches
1									
3									
6	74.8	76.9	1.42	1.57			1.104	6.0	6.6
12	74.5	76.6	1.40	1.55			1.107	10.4	11.5
18	74.2	76.3	1.37	1.51			1.101	13.0	14.3
24	74.0	76.1	1.35	1.49			1.103	15.1	16.7
30	73.8	75.9	1.33	1.47			1.105	16.8	18.6
36	73.6	75.7	1.31	1.45			1.107	17.4	19.3
42	73.6	75.7	1.31	1.45			1.107	18.1	20.0
48	73.5	75.6	1.30	1.44			1.108	18.2	20.2
54									
60									
66									
72	72.9	75.0	1.25	1.39			1.111	18.7	20.8
78									
84									
96									

Computed by JEH Checked by AGK

FIGURE 47.—*Example of method used to adjust and transpose storms.*

justment of the storm center to the French Broad watershed would result in an increase of about 11 percent over the total amounts experienced in the storm.

In order to determine the representative *observed* dew points for this storm, the weather maps for this period and the records of those stations which were well within the inflowing current of moist air were studied. The results of this study showed the *average* dew point for various durations to be as listed in column 2 of figure 47. The maximum expected dew points for various time intervals of persistence in mid-July were taken from the comprehensive study which was made in connection with the maximum storm studies. These maxima are entered in column 3.

The next step was the determination of the effective precipitable water, "W_e," per unit column of air for the observed dew point and for the maximum expected dew point, using figure 46. To do this, the height of the inflow barrier of the observed storm and the height of the inflow barrier of the Upper French Broad River Basin were determined. The average heights to which the bases of moist air columns had to be lifted to enter the rainfall area were integrated through the inflow width, using topographic maps prepared by the Civil Aeronautics Authority. In this case, the inflow barrier to the observed storm area was 1600 feet while the most critical inflow barrier to the area above Asheville was 2000 feet. These heights were recorded in appropriate places on figure 47, and were used in determining the effective precipitable water, "W_e," from figure 46. These values were recorded in columns 4 and 5, respectively, of figure 47. The ratio R of the maximum effective precipitable water, "W_e (max)," to the observed effective precipitable water, "W_e (obs)," shown in column 6, gives the change required to adjust the storm to its physical upper limit for moisture charge only when transposed to the drainage area above Asheville. This change amounted to an increase of about 11 percent. Having established the ratio required to effect the adjustment of the storm to its upper limit, it was only necessary to determine the observed time-depth of average rainfall for selected durations and multiply these values by the ratio in order to find the maximum depth for the same durations. The observed average storm rainfall depths for various durations are shown in column 7, and the results of adjusting these amounts to the physical upper limits for this storm appear in column 8. The observed and adjusted depth-duration curves for the storm of July 13-17, 1916, are shown in figure 48.

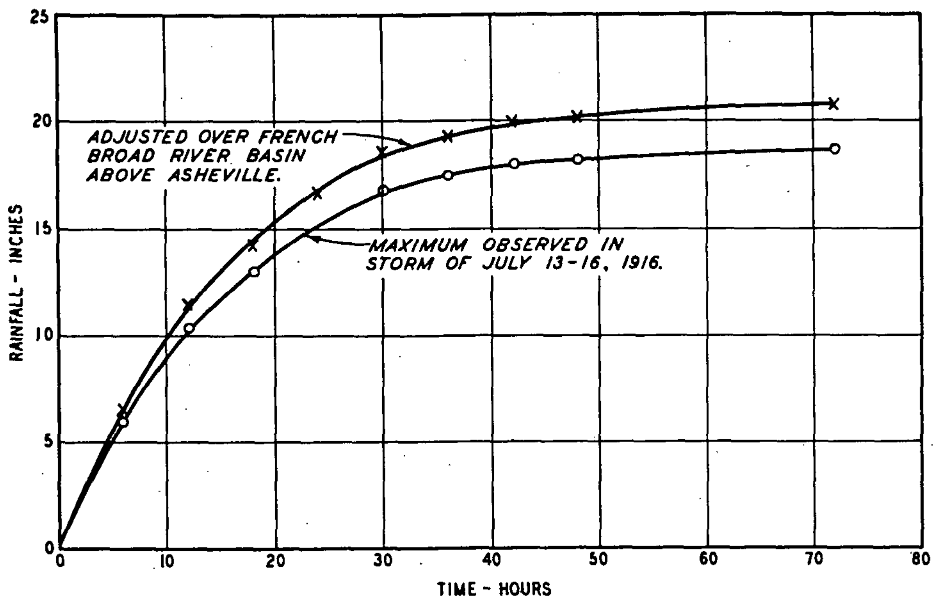
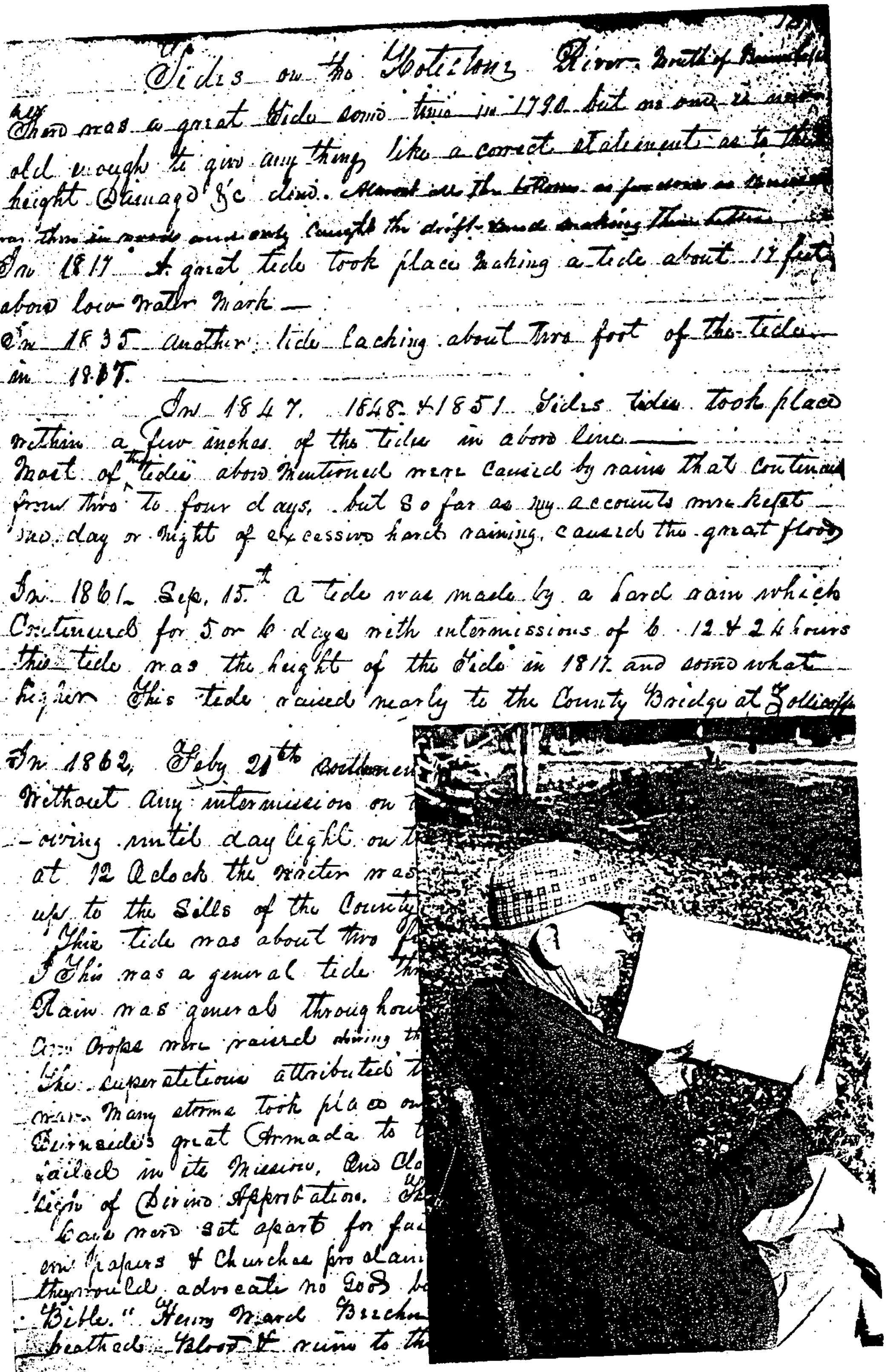


FIGURE 48.—*Observed and adjusted depth-duration curves for storm of July 13-16, 1916.*



Tides on the Holston River South of [illegible]
There was a great tide some time in 1790 but no one is now
old enough to give any thing like a correct statement as to the
height Damage &c done. Almost all the bottoms as far down as [illegible]
was then in woods and only caught the drift [illegible] making their [illegible]
In 1817 A great tide took place making a tide about 12 feet
above low water mark —
In 1835 Another tide lacking about two feet of the tide
in 1817.
In 1847. 1848 & 1851 Tides tides took place
within a few inches of the tide in above line —
Most of the tides above mentioned were caused by rains that continued
from two to four days, but so far as my accounts were kept
no day or night of excessive hard raining, caused the great flood
In 1861 Sep. 15th a tide was made by a hard rain which
Continued for 5 or 6 days with intermissions of 6. 12 & 24 hours
this tide was the hight of the tide in 1817 and some what
higher This tide raised nearly to the County Bridge at Zollicoff[er]
In 1862, Feby 21th [illegible]
Without any intermission on [illegible]
-ving until day light on [illegible]
at 12 O clock the water was [illegible]
up to the Sills of the County [illegible]
This tide was about two [illegible]
This was a general tide [illegible]
Rain was general throughou[t] [illegible]
Crops were raised during [illegible]
The superstitious attributed [illegible]
war. Many storms took place on [illegible]
Burnside's great Armada to [illegible]
failed in its Mission, One [illegible]
Sign of Divine Approbation. [illegible]
Days were set apart for fa[sting] [illegible]
our papers & Churches proclaim[ed] [illegible]
they would advocate no God b[ut] [illegible]
Bible." Henry Ward Beechu[r] [illegible]
heathen Idols & ruins to th[e] [illegible]

FIGURE 49.—*Personal flood records—reproduction of timeworn page from Fickle Scrapbook with inset showing Cross Diary being read by its elderly author.*

CHAPTER 3

SOURCES OF FLOOD DATA

The basic planning for any multiple-purpose river basin development, such as that for which the TVA was made responsible, demands among other things a thorough familiarity with the flood-producing characteristics of the streams in the Basin.

The preceding chapter describes a number of the great flood-producing storms of record in the Tennessee Valley Region. Several of these storms, particularly those before 1900, occurred long before an adequate network of stream gaging stations was established in the Valley. As a result, official records of floods for those early years were scant and in many cases non-existent, and it became necessary to engage in intensive historical research throughout the Valley in order to provide the information needed.

This chapter 3—after summarizing the uses for which flood experience data are required, particularly as they apply in the Tennessee Basin—discusses Valley flood history investigations and sources of both historical and recorded flood data. Flood profiles, described briefly at the end of the chapter, are discussed in detail in chapter 4.

USES OF FLOOD EXPERIENCE DATA

Flood experience data are put to many uses. For example, in the location and design of any structure along a stream, knowledge of the height, discharge, and frequency of occurrence of floods on the stream is essential. The determination of required spillway capacity for safety of dams and of required storage capacity for adequate flood control makes it imperative that the flood-producing characteristics of the stream be explored exhaustively. The highway and railroad engineer needs to know how high to build his roads to keep them above flood danger and what volume of water he must allow for in his bridge openings. Large and small industries, investigating attractive plant sites in the wide, flat river bottoms, are faced with similar problems of keeping floor heights above flood level and designing protective works. Many industries in the Valley have suffered heavy flood losses because of inadequate knowledge of past floods on an innocent-appearing stream nearby. Along the Tennessee River and on its navigable tributaries, the determination of navigation clearances under bridges and transmission line crossings requires a knowledge of flood experience on the streams. Finally, the benefits to be derived from flood control projects, whether by up-river storage or by flood walls and levees, can be determined only through research into the height, frequency, and effects of flooding in the past.

FLOOD HISTORY INVESTIGATIONS

The searching out of historical information on floods in a watershed is a demanding job that requires the engineer to be a skilled researcher, a tireless and persistent investigator, and at the same time a strong and healthy field man. As one of TVA's engineers said after completing a flood history investigation in the rugged Hiwassee River watershed, "the one doing the job should be very much of an optimist, humorist, hiker, laborer, farmer, and engineer, as well as an interpreter of Indian signs and languages." In the field he must be both a detective and diplomat who can search out witnesses, pry into family histories, gain access to personal diaries and records, and still maintain friendly relations with the people he interviews. In one case on the Clinch River the persistent field investigator even tried to obtain permission to remove the wallpaper in a room to uncover an old high water mark.

Coupled with these qualities he must have the ability to sift and weigh evidence, and to explain and reconcile apparent discrepancies in the data.

Sources of data

There are three major sources of flood history information. These are: (1) published records and reports, including newspapers; (2) unpublished records and reports; and (3) information obtained through interviews with local people. All are valuable sources of data and none may be ignored in a thorough investigation into the history of floods.

In the Tennessee Valley investigation, published data include official technical records of the U. S. Geological Survey, the U. S. Weather Bureau, the State Division of Geology, the U. S. Corps of Engineers, and other government agencies, together with special reports by these agencies on individual storms. TVA has prepared and published many such special reports, as have the U. S. Geological Survey, the U. S. Corps of Engineers, the U. S. Weather

Bureau, and others. In addition, private organizations occasionally investigate and publish reports on great floods which have particularly affected them. An example is the excellent report, *The Floods of July 1916*, issued by the Southern Railway Company in 1917. A prolific and valuable source of published flood information is the files of the newspapers of the region. This source of data is discussed later in this section. Finally, there are the technical magazines carrying stories on the effects of great floods, and books describing the region. In this latter category are the published accounts of early explorers and travelers in the area. These accounts, often written in the form of a log or diary, mention delays caused by unusual rainfall or high stream stages that are indicative of floods on the stream being studied.

The flood history investigations of the U. S. Corps of Engineers in the Valley proved very useful. During a period of more than 60 years, beginning in about 1870, this agency made intensive surveys on the Tennessee River and its tributaries leading toward the development of navigation, water power, and flood control in the Valley. The Corps of Engineers' profiles of river bed, low water, and river banks were used as basic data for many of TVA's flood crest profiles. Corps of Engineers' river mileages were adopted wherever they were available. Information on high water marks located by the agency was included with the much more extensive data collected by TVA in preparing flood crest profiles. Field books used by the Corps of Engineers were searched for information not contained in published reports.

Unpublished data of great value were found in the personal records of local people (fig. 49), written in diaries, family Bibles, and account books. The discovery of these unique records was kept constantly in mind by the investigators as they interviewed residents along the streams for flood information. A similar source was the private records kept by industries, commercial establishments, municipalities, utilities, and others directly affected by floods. Unpublished reports of flood investigations by private organizations or government agencies were also a source of data.

The great bulk of information on flood heights, particularly in the rural areas, was obtained by interviewing those who actually saw one or more floods or who had information passed down to them by older people. Among those who witnessed floods, some had made a mark on a house, mill building, barn, tree, or rock which was still visible, while others depended entirely on memory of how high the flood crest was in a house, or how far it reached in a field, along a road, or among the trees in an orchard. Dates were fixed in relation to some great event, such as a war, or by a birth, marriage, or death in the family shortly before or after the flood.

The intensity with which interviews with local residents were carried on depended on the time available and the importance of the stream from a flood-producing standpoint. Theoretically, every family living on the river should be interviewed, and old residents who have moved away should be located. During the Valley investigations much effort was saved through preliminary discussions with county agents, local postmasters, or others familiar with the neighborhood. Lists of old residents were assembled from these discussions and newcomers to the area were eliminated from consideration. Valuable leads to unique records or diaries were sometimes developed. There was a considerable advantage to the investigator in knowing beforehand who lived in each house and which member of the family was most likely to have flood history information.

An important fund of information on crest heights and the effects of overflow was available in photographs taken by amateurs and professionals during floods. A productive method used by TVA engineers in uncovering such evidence was through the insertion of advertisements in the local newspapers requesting flood pictures. Usually, the newspaper editor was willing to convert this advertisement into a news story, expanding it to explain the interest of TVA in the information, and thus paving the way for the investigator as he traveled through the area. All professional photographers were contacted and their files inspected for flood views, and a similar search was made at newspaper offices. Examples of pictures from various sources are figures 50 through 53; figure 50 showing a Chattanooga business street during the 1886 flood was obtained from a private collection; figure 51 showing the January 1918 flood at Clinton, Tennessee, is typical of pictures obtained from private individuals; figure 52 showing the Duck River flood of February 1948 at Columbia, Tennessee, was taken by a commercial photographer; and figure 53, an aerial view of the same 1948 flood at Shelbyville, Tennessee, was obtained from newspaper files.

Information of considerable value in interpreting old flood data was often obtained from published and unpublished histories of the region. The date of settlement of the area helped determine how far back information on floods might be expected to extend. The dates of construction of certain projects helped explain apparent discrepancies between marks of current and past floods. For example, the construction of a highway or railroad fill along a river reach may greatly reduce the extent of overflow and raise the flood crest level in the reach. In a growing city, the gradual increase of encroachments on the flood channel will change substantially the relation of flood height to peak discharge.

Reliability and accuracy of sources

In general, the reliability and accuracy of the sources named and discussed in the preceding paragraphs will decrease in the order in which they are listed. That is, published technical records may be expected to be more reliable than unpublished data since they have presumably been reviewed carefully before publication. Similarly, unpublished technical

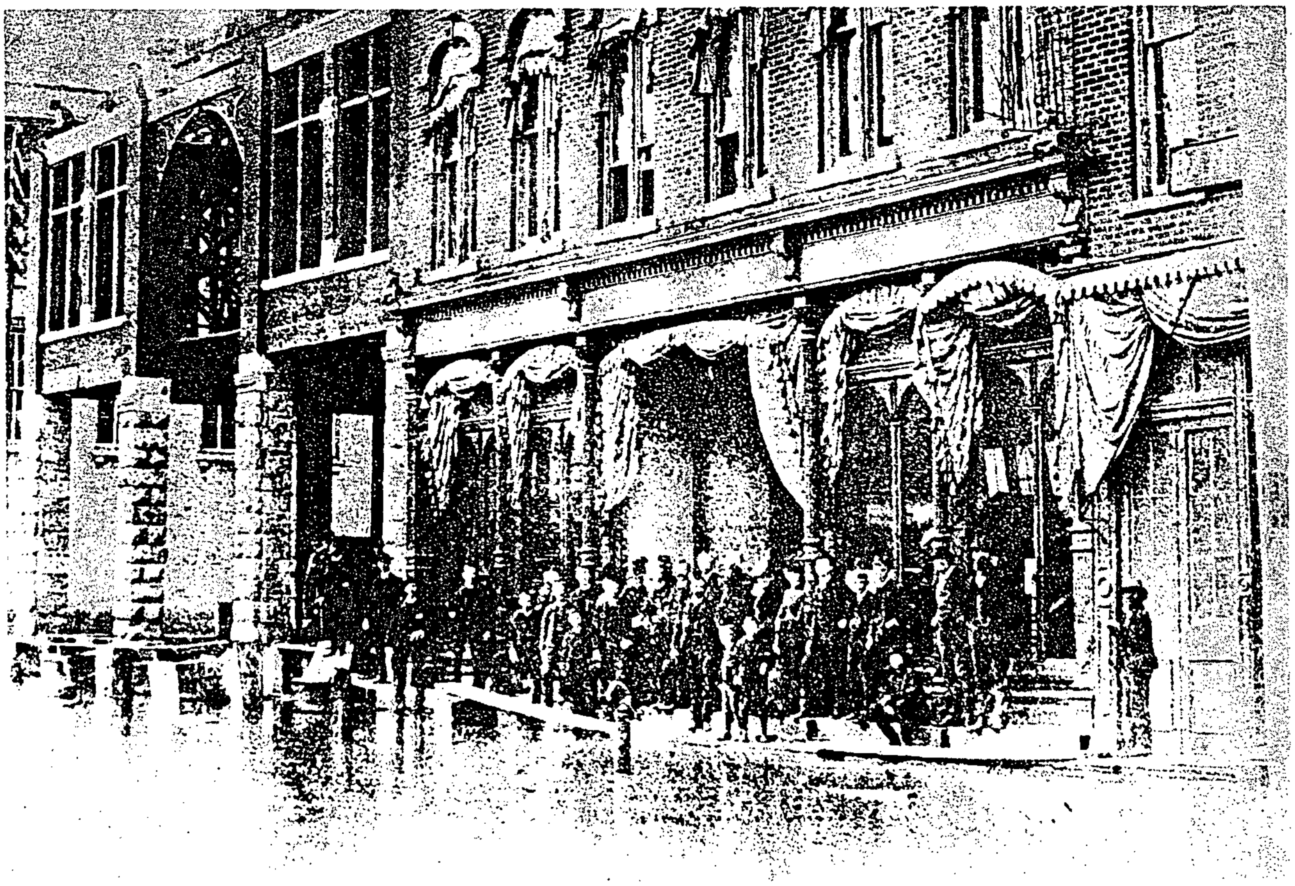


FIGURE 50.—*March 1886 flood at Chattanooga showing water in front of Loveman's old store (example of picture from private collection—Cline Photo Collection, Chattanooga).*

FIGURE 51.—*Clinch River flood of January 1918 at Clinton, Tennessee (example of picture by private individual).*

FIGURE 52.—*Flood of February 1948 at Columbia, Tennessee (example of picture by commercial photographer —Camera Shop, Columbia).*

FIGURE 53.—*Flood of February 1948 at Shelbyville, Tennessee (example of aerial view obtained from newspaper files —Nashville Tennessean, Robert C. Holt, Jr.).*

data that have been recorded soon after the occurrence of the flood or that are based on research by skilled investigators may be expected to excel in accuracy the memories of local residents.

Published records and flood accounts, however, cannot be accepted as fact merely because they appear in print. This is especially true of newspaper accounts which are sometimes exaggerated for effect or which may be based on false reports without subsequent publication of corrections. Even streamflow data which, at the time of publication were thought to be entirely reliable, may be revised considerably as a result of subsequent measurements.

Flood heights marked by local people at the time of occurrence are desirable forms of flood history data. However, unless the dates have also been recorded, these marks may lead to erroneous profiles because of confusion in remembered dates. This situation is most likely to arise when two large floods occur in the same year, as on the French Broad River in August 1940, or in successive years, as on the Clinch River in 1917 and 1918.

High water marks based on an observer's memory or on hearsay may turn out to be appreciably above or below the actual flood crest level. However, many marks of this type for floods occurring as much as 100 years ago have been found to agree very well with other data when the profiles are drawn. During the flood history investigation in the Little Tennessee River watershed, an 86-year old farmer living near the mouth of Tuckasegee River told the investigators of a great flood in 1840 that his father had described to him. The crest level, at the door sill of a spring house, was recorded and, as the party moved on up the Tuckasegee River, the clue to this old flood was followed up. Four more marks, all based on stories handed down by older people, were found along the lower 50 miles of the river. Only one of these was a definite mark. A profile was sketched through these marks with considerable misgiving since it seemed unreasonably high compared to profiles of other floods. Four years later, the great storm of August 30, 1940, produced a flood whose profile exceeded that of 1840 in the upper reaches and followed almost exactly on it in the middle portion. Thus, the almost legendary story of the old flood was confirmed on its one hundredth anniversary.

Care was taken to record the essential facts of the observer's story regarding each mark so that an estimate of the accuracy of the mark could be made. Sometimes, information which appeared irrelevant early in an investigation became valuable supporting evidence when the complete picture of floods on the stream was viewed.

Flood history reports

For most effective utilization, the flood history data were assembled into reports for each stream which included brief descriptions and histories of the watershed, tabulations of official flood records, plotted hydrographs, descriptions of individual floods, lists of high water marks, profiles of high and low water, and location maps. Appended data included abstracts of published information, photographs, and other supporting records. On some streams, reports have been prepared on intensive investigations of short reaches through urban and suburban areas where flood problems exist or may develop. These reports contain more detailed data than the general flood histories. The flood history reports interpret the data insofar as this is possible and present them in such form that they can be used to meet the many needs for this type of information in a water resource program.

Damages

In studies of flood control projects, the relative importance of floods of the past may depend to a large extent on the damages they caused or would cause under present conditions. High floods on streams flowing through rugged, unsettled country are often of less interest than smaller floods on streams draining highly developed areas, except to the extent that they contribute to damaging overflows downstream. For example, a relatively small summer flood over the intensively cultivated bottoms of the French Broad River above Asheville may cause damage as great as that resulting from a relatively large flood on the Little Tennessee River above Fontana Reservoir. Thus, the collection of data on damages becomes an essential part of a flood history investigation.

Files of local newspapers are the principal source of information on damages caused by past floods, except where engineering reports have been prepared soon after the flood. Data on damages resulting from bridge, highway, and railroad washouts have been obtained from the records of the organizations concerned.

HISTORICAL FLOOD DATA

Systematic observation of stream stages on Tennessee River tributaries did not become widespread until well into the twentieth century. To obtain information on the great floods of the past, some of which proved to be the greatest known, it was necessary to depend largely on non-engineering observations and records. Of these, the ones that gave most satisfactory results were newspaper accounts, personal diaries, and interviews with witnesses of floods.

Newspaper accounts

Floods were prime news to the editors of early newspapers in the Tennessee Valley. Whole pages were devoted to descriptions, human interest items, and correspondents' reports on major floods. Issue after issue of the city dailies carried items on the flood as they came in from outlying areas. Figure 54 is a reproduction of the first page of *The Journal and Tribune,* Knoxville, Tennessee, for May 23, 1901, carrying the first complete news of the devastating

THE JOURNAL

VOLUME XV. NO. 33, KNOXVILLE, TENN., TH

MILLIONS OF DOLLARS DAMAGE DONE IN UPPER
MANY HOUSES, BRIDGES, MILLS WASHED AWA

Southern Railroad is Badly Crippled East of Johnson City Station.

Many Days May Elapse Before Trains Can Be Run On Main Line.

Passenger Train Plunges Through Water Up to Car Steps on Asheville Line.

The Southern railway at this point is completely cut off from the east, as no trains can get beyond Johnson City on the main line by way of Bristol, on account of the washing away of the bridge over the Watauga river, while the giving way of bridges and washouts on the line between Asheville and Salisbury have also closed that line. The only way of reaching the east, for passengers, mail or express matter, is either by way of Harriman and Cincinnanti, over the Cincinnati Southern railroad, or by the roundabout way of Ooltewah Junction, Atlanta, and the main line by way of Charlotte and Lynchburg.

Consequently, all trains east of Asheville on the main line, and beyond Johnson City, on the line via Morristown, have necessarily been annulled for the present, and no through business of any kind is passing over the Southern main line through this city, all of it being diverted at Chattanooga or other points west, to the line by way of Atlanta and Charlotte.

To Span the Watauga.

Superintendent Ewing, Trainmaster Westcott, Assistant Engineer Bernard and Roadmaster J. E. Platt went to Johnson City yesterday morning for the purpose of inspecting the tracks beyond Johnson City, and trying to devise some plan for spanning the Watauga river with a temporary structure as soon as possible after the flood there subsides. None of these officials had returned to the city last night, and there was no one in authority who would venture any opinion as to the time when traffic via Bristol with the east might be resumed. It was thought that it might be a month before even a temporary bridge could be completed.

River at Knoxville Will Pass the Danger Line at Early Hour Today.

Big Bridge Thronged With Large Crowds of Sightseers Yesterday.

Figures on Previous Famous Floods That Put the City in Darkness and Did Great Damage.

The flood tide is on the Tennessee river at this place and the indications are that not only will the danger line of twenty-nine feet have been passed by this morning, but that the river will have reached at least thirty-five feet, and perhaps even higher.

From eight feet Tuesday afternoon the river rose steadily all night at the rate of eight inches an hour and by yesterday morning registered eighteen feet above low water mark. It continued to rise until noon, when the guage showed nearly twenty-one feet. From that hour until 5 p. m. it rose steadily and had reached twenty-six feet.

At twenty-one feet the water had backed into First creek and covered the mill dam near Front avenue.

At twenty-six feet the water reached the steps at the local company's warehouse and was just lapping the southside of Front street near the creek. The steamer Flora Swann, which usually anchors at a distance of fifty feet from the warehouse, that being just within the river bank, was hugging close to the warehouse yesterday afternoon in water three and four feet in depth. The two King boats were anchored in what is ordinarily the highway above the river banks. Big sand barges were stationed almost in the back yards of houses along the river front.

River Men Notified.

Early yesterday morning, when the news of the threatened flood was read in The Journal and Tribune, the owners of lumber and other property began to work to save their property by removing it to places of safety. Large forces of hands were pressed into service and the

SUMMARY OF TH

The torrents which rushed through the have left ruin and disaster in their wake in m rough estimates of the total damage, although to have been lost.

The railroads of this territory are the gr many thousands of dollars, exclusive of the del delay. The loss of the large railroad bridge ov ands. It was the longest and most expensive ste railway, and was thought to be high enough to at Embreeville is also reported gone, and severa between that city and Salisbury. The Atlanta, bridges beyond Blue Ridge, Ga., and all the ro damage to tracks, culverts and roadway.

It is possible that the damage of this cha the east will subside today, the Tennessee is like haps later, and this may cause trouble further s the Hiwassee river, near Charleston, west of the receded by nightfall.

By the breaking of a dam across the I county, was the worst sufferer. Elizabethton is white settlement in Tennessee, one hundred and settlement.

The town is at the juncture of the Wat great Appalachian range, and have for their tri that also have their source in these great mount sea level. These two rivers, with their tributa tending from Elizabethton to the divide, where Carolina to the Atlantic.

The rain storm extending over this wide lence. The rain fell and rushed in torrents do into the two rivers, then ran on converging li the disaster there when the dam broke. The the streams with a rapidity irresistible, and the creases in volume. In the history of the town quarter, there has never before been anything li

The damage at Elizabethton will be $20(

The Chucky river claimed three victims at least half a million dollars.

In Carter county it is estimated that the French Broad is reported, and heavy damage to

FIGURE 54.—*First page of Knoxvi*

ND TRIBUNE.

PRICE 2 CENTS
EVERYWHERE except on Trains and Sundays—FIVE CENTS.

AY, MORNING MAY 23, 1901. Price: Daily, 2 Cts.; On Trains and Sundays, 5Cts.

T TENNESSEE BY RAGING, ROARING FLOODS; D LIVES SACRIFICED—ELIZABETHTON WRECKED

OD'S DAMAGE.

streams on Tuesday afternoon and night, ties, and it is yet too early to make even in into the millions. Five lives are known

gle sufferers, and their losses will foot up ffic and the loss of business incident to such atauga alone, will amount to many thous- on the Knoxville division of the Southern e effects of any possible flood. The bridge are washed away on the Asheville division, e & Northern has also lost several small s section have suffered from washouts and

not yet over, for, while the high water to tinue to rise at least until noon, and per- was reported yesterday that the bridge over s in danger, but the water is said to have

, Elizabethton, the county site of Carter where James Robertson established the first ears ago. It was known as the Watauga

the Doe rivers, both of which rise in the numerous creeks, some of them large ones, e of which rise six thousand feet above the a very large area of mountain country, ex- rs flow eastward through the state of North

mountain country was one of unusual vio- mountain sides into the numerous creeks and ing and commingling at Elizabethton, hence fall is so great that the waters come down force as the current accumulates and in- ing through a period of a century and a

nly one life was lost.
eneville and did damage in Greene county of

reach a round million. One death on the hich will be a total loss.

Estimated that the Property Loss in the County Site of Carter County Will Be Two Hundred Thousand Dollars—Only One Life Lost There.

Million Dollars Damage in Carter, Half a Million in Greene County; Three Children Are Drowned Near Greeneville.

Many People Rescued From Their Houses Along the Chucky River. Heavy Damage at Watauga, Chestoa and Devault's—White Store Suffers—French Broad Plays Havoc With Fine Crops—Several Cases of Drowning Reported.

Special to The Journal and Tribune.

Bristol, May 22.—Details from Elizabethton are slow to come in, all the local wires being down and all bridges washed away.

It is known about seventy-five residences and all bridges at Elizabethton including the county iron bridge and the Virginia and Southwestern railway's iron bridge over the Watauga, were swept away.

The only life lost at Elizabethon was that of a negro but the whole valley is a scene of desolation, houses, fences, barns, crops, etc., being gone. Factories were also damaged and the loss may exceed $200,000 at that point.

—:—

Heavy Damage at Watauga.

The damage at Watauga, on the Southern railway, six miles south of Elizabethton, will probably be $150,000. The Southern railway's splendid iron bridge there was swept away entirely. Ten loaded freight cars went down with the bridge into the tide twenty-five feet above the normal surface. Three lights are reported on the bridge when it went and it is feared three men might have perished. Plummer's large flouring mill and 25,000 logs of the Watauga Lumber company were swept away at Watauga. The Watauga tannery was also flooded and partially destroyed.

—:—

Chestoa and Devault's.

The dam at Chestoa is still standing. The immense flood washed out the mill

his house that his family had to be rescued by a boat.

Many other narrow escapes from drowning are reported. James Johnson, a farmer who resides five miles from Greeneville, was in Knoxville attending the farmers' convention yesterday, but hurried home today when he read of the flood news. His home, which is a two-story brick structure, was submerged in water up to the second story. When Mr. Johnson arrived he was given the information that his mother, wife and a lady guest had [illegible] fely taken from the house from the second story window in a boat.

Three children of Joe Hill, a negro farmer, living seven miles from Greeneville, were drowned yesterday. Hill was at home and seeing that the water was rising rapidly went to his barn to save his stock. So rapidly did the river come up that before he could return to his home his three children, the youngest of whom was 13, were drowned.

Competent judges place the entire damage in Greene county at not less than half a million dollars.

At last reports tonight the Chucky was still rising, but very slightly. It is believed that all damage has been done that can be by the present flood.

FRENCH BROAD RUINS FINE CROPS

Man Named Bolivar Drowned in the Chucky at Leeper's Mill.

Special to The Journal and Tribune.

nessee, newspaper—May 23, 1901.

flood of May 21-22, 1901, on the South Holston and Watauga Rivers.

Interviews with the ever-present "oldest resident" were often reported, including in many cases comparisons with earlier floods. Useful information thus was obtained, not only on the height of the current flood but of others that pre-dated newspaper files. For example, eleven days after the record flood of July 16, 1916, in the French Broad River watershed, the Asheville, North Carolina, *Citizen* carried an interview with a pioneer resident that afforded valuable clues to early floods. The story listed four historical "freshets," namely, the April freshet of 1791, the May freshet of 1845, the June freshet of 1876, and the July freshet of 1916, and described the pioneer, Mr. W. J. Alexander, as "the greatest living authority when it comes to 'freshets' in this part of the state. Mr. Alexander was born in 1830, fifteen years before the first notable freshet which devastated Buncombe County. His knowledge of the previous great freshet was imparted to him by his grandfather, James Alexander. . . ." The grandfather lived in the Swannanoa River watershed.

Regarding the flood of 1791, the article states that it must have been even greater than the flood of July 1916. "Mr. Alexander says the freshet was a terrific one and that all through the Swannanoa Valley there was naught but a sea of turbulent fury. Everything in its path was swept clean, and the waters have left their marks in the Valley to this day. This was a long, long, long time ago—125 years ago, and there was no Asheville here then to damage."

The area was still largely undeveloped at the time of the 1845 flood. The article states that there was still "no Southern Railway passenger station and the May 'freshet' of that year (1845) didn't interfere with the operation of trains one particle." The Southern Railway suffered enormous losses in the 1916 floods. " 'The freshet of 1845 covered all of the bottom lands,' says Mr. Alexander, 'but the waters receded early enough for the farmers to plant their crops, or to replant them . . . The freshet washed great holes in some of the farms, but it filled up other great holes, too,' remarks Mr. Alexander, who has lived long enough to become a philosopher."

The clue to the 1791 flood was followed up, and information was found confirming the newspaper story and indicating that the ancient "freshet" was 4 to 6 feet higher on Swannanoa River than the great 1916 flood.

Other newspaper articles, old diaries and data obtained from old residents made it possible to add 100 years to the official record of floods on the French Broad River at Asheville and 130 years to the official record on Swannanoa River. This typical example has been duplicated a number of times in the flood history work of TVA. During a 1944 investigation of floods on Beaver Creek at Bristol, Tennessee-Virginia, for instance, it was possible from newspaper files, old photographs, and interviews with residents to develop a fairly complete list of all important floods occurring since 1867 and to make an estimate of the height of each one. Streamflow records on this stream covered only 19 months of the 77 years included in the flood history. This experience has been duplicated in many other cases.

Files of old newspapers are available in newspaper offices, public and university libraries, collections of historical societies, and in the libraries of private individuals. These collections, as a rule, extend farther back and are more complete for the large city papers than for the small town publications. In some cases, however, small community newspapers have remained in family ownership for generations and files of the papers have been maintained with great pride. Many small town publishers had an unfortunate habit of selling out after a few years and taking their accumulated files with them. Unless these files fell into the hands of some library or other organization interested in preserving them they were soon lost or destroyed. Even some of the large city newspapers suffered the same fate. The file of Knoxville, Tennessee, papers in the Lawson McGhee Library, beginning in November 1791, has long gaps near the beginning and end of the nineteenth century. The Chattanooga, Tennessee, library collection is only fragmentary prior to the Civil War.

Correspondents' reports to the large city dailies often provided nearly as much flood news as was contained in the local weekly paper in the flood area and in some cases the only news. The Knoxville, Asheville, and Bristol papers covered the east half of the Valley, while the Chattanooga, Nashville, and Paducah papers carried reports on the western half. A flood occurring a day or two after a weekly paper was issued was stale news for the next weekly edition, but the story was usually forwarded immediately to the nearest daily paper. Most of the available information on the great floods of May 1856 in Middle Tennessee came from correspondents' accounts published in a Nashville paper.

Personal diaries

Perhaps the best known of the personal records from which flood history information was obtained was the Fickle Scrapbook. Robert P. Fickle, who died near the turn of the century, was a citizen of Sullivan County, Tennessee. He lived on South Fork of Holston River about 3 miles below the mouth of Beaver Creek in the center of the area bounded by Bristol, Kingsport, and Johnson City, Tennessee. Mr. Fickle was deeply interested in scientific and historical matters, and he gathered together much material on these subjects. Upon his death at the age of 73 he left at least a portion of this accumulated material in the form of a scrapbook.

Figure 49 shows a reproduction of a page from the scrapbook. Four of its pages were headed "Tides on the Holston River, Mouth of Beaver Creek." In old-fashioned handwriting the account begins:

There was a great Tide sometime in 1790 but no one is now old enough to give anything like a correct statement as to the height, damage, etc., done . . .

In 1817 a great tide took place making a Tide about 17 feet above low water mark.

In 1835 another tide lacking about two foot of the tide in 1817.

In 1847, 1848, and 1851 Tides took place within a few inches of the tides in above line. . . .

Speaking of the flood of March 1867 Mr. Fickle wrote:

The greatest tide yet in Holston River was caused by the most unprecedented raining season known to the oldest people.

After describing carefully the occurrence of almost continuous rainfall which lasted from February 26 until March 7, 1867, and the several rises that accompanied it, Mr. Fickle reported that:

The greatest height of water on Thursday 7th was 27 feet 4 inches at the mouth of Beaver Creek above low water mark.

Great damage was done on the rivers and creeks. All the narrow Bottoms on the River was damaged, either by being washed into holes and gutters or covered by sand. Many mills were carried off, also houses, Barns and Stables, thought to be out of reach of high water.

That the 1867 flood was unprecedented in the period of white settlement of the country is indicated by the following remarks of Mr. Fickle:

In many places . . . Indian towns and grave yards were exhumed by this tide—the history of which is unknown to any person or no one has a tradition thereof.

By the wash at River Bend forge . . . a number of skeletons were uncovered. At the mouth of Watauga, on the south side, the washing uncovered a large Indian Town that had been covered up by some great flood. The wigwams and fire coals, spikes and Hatchets, and Pottery attest a race long since extinct. . . ."

The March 1867 flood was the greatest known on the upstream half of the Tennessee River, and it was either a maximum or a major flood on most of the upstream tributaries.

A personal diary covering floods more recent than those in the Fickle Scrapbook is that of Staples Cross, approaching 90 years old, who has lived on Poplar Creek near the Anderson-Roane County line (Tennessee) since 1913. Poplar Creek is a tributary of the Clinch River. Mr. Cross has kept a personal diary of local events the greater part of his adult life. The insert in figure 49 shows him reading this diary of which the following are excerpts regarding high water on Poplar Creek:

June 28, 1928—Poplar highest I ever seen it.

March 23, 1929—High waters in general. But not quite so high as in June 1928.

September 29, 1944—High water in Poplar Creek. Rained all night on the 28th. Rained all day on the 29th. Liked about 2 feet being as high as it was once before—about 15 years ago.

Another source of pre-record flood data of a similar nature was the "Records of the Moravians in North Carolina," covering the period 1752 to 1792. These journals do not refer directly to portions of the Tennessee Valley in Western North Carolina but are concerned with the vicinity of the Yadkin River near Winston-Salem, North Carolina. However, they do contain material which is indicative of large storms and floods in Western North Carolina on a number of occasions.

Interviews with witnesses

Interviews with local people who actually witnessed floods or who have information on flood heights passed on to them by older people are a basic source of pre-record data. They provide the only information on floods in the long reaches between towns and gaging stations. Flood marks shown by these people are the basis for the development of profiles from which high water elevations at any point can be determined.

Interviews with witnesses are not as complete a source as old newspaper files, from the standpoint of flood frequency. Whereas, newspapers may carry stories on any out-of-bank stage, people living along the river will recall only the very large floods, or those which particularly affected them. Thus, in a flood history extending back for 80 or 100 years, enough marks to plot profiles over considerable reaches may be available for only two or three floods.

In his discussions with those who witnessed early floods, the investigator is faced with the problem of spurring memory associations without at the same time asking leading questions. He cannot ask a witness if his mark is for the flood of 1906, for instance, but must try to relate the incident to some outstanding occurrence that can be dated. He must himself be cognizant of certain aspects of local history, such as dates of construction of railroads, highways, or bridges to which the witness may relate the flood year.

Experience has shown that witnesses have much more difficulty recalling flood dates than they have in remembering with fair accuracy the height that a flood reached. Usually, the point the water came to is referred to some familiar object about the home or business and the story is told over and over again to friends and family. The principal danger to be guarded against is the habit of exaggeration which moves the flood higher and higher as the years pass by. From time to time in the Tennessee Valley investigations, marks have been pointed out in all seriousness which plotted as much as 10 feet above authenticated profiles for a given flood.

The stories of the witnesses are summarized by the engineer and made part of the description for each mark obtained. Some typical examples of these descriptions are given below. It will be noted that some are for actual marks made by the witness, some are remembered water levels, and some are based on information handed down by others.

French Broad River, Mark No. 91, May 1901—M. C. Atchley, operator of Atchley's Mill, Rankin, Tennessee, showed knife cut marks made by him and his brother on a dust spout in the mill for the flood crests in 1901 and 1902. The mark for the 1901 flood was 8¼ inches above the mark for the 1902 flood. Mr. Atchley stated that the flood of July 1916 did not quite reach the main floor of the mill and was probably three feet below 1902 at this point. Other local residents agree that the floods in 1901 and 1902 were the highest in the memory of anyone now living in the vicinity. A flood is known to have occurred in the 1860's which was not quite equal to these two floods. This early flood was said to have been worse on Pigeon River.

Holston River, Mark No. 12, March 1867—C. A. Mooney, farmer, R.F.D. No. 1, Rogersville, Tennessee, pointed out a notch cut in the steps inside the old Shepherd Mill, and the words "March 7, 1867" inscribed nearby. Mr. Mooney said that this notch marked the flood crest elevation and that it was put there by the former mill operator at the time of the flood. This story was verified by questioning other people in the neighborhood.

Little Tennessee River, Mark No. 95, October 1898—Sam Hall, Franklin, North Carolina, age 86, who has lived all his life on the Little Tennessee River, said he had marked all the old floods on a sycamore tree on the river bank. The last and largest of these floods that he marked was the one in October 1898. He recalled that the mark was 10 feet above the ground. Examination of the tree disclosed several dim gashes, and Mr. Hall pointed out the one he thought to be for the 1898 flood. This mark measured exactly 10 feet above the ground. Mr. Hall could recall nothing about the other floods.

The following brief reference was abstracted from the Franklin, North Carolina, *Press* for October 12, 1898:

> Mr. Sam Hall says he has marked all the high waters since February 1875, which he says was the highest within his recollection till October 4, 1898, which was 10 inches higher than the freshet of 1875. (The flood history investigators talked to Mr. Hall 40 years after this article was written.)

Nottely River, Mark No. 10, September 1898—Luther Kisselburg, Culberson, North Carolina, age 60, lived on Nottely River just upstream from North Carolina Highway No. 60 bridge until 29 years ago when he moved to Culberson. He still owns and looks after the old home on the river. Mr. Kisselburg said he believed that the September 1898 flood was the largest in volume of water, although the floods of April 1936 and July 1938 were higher at his home site. He thought this was because the bridge and approach fills 700 feet downstream were different now than in 1898.

The 1898 flood, according to Mr. Kisselburg, just reached the top of the next-to-the-top stone step in front of his house. He recalled that the flood occurred a short time before he went to Kansas on election day when he was 20 years old. He was born in 1878.

The highest flood Mr. Kisselburg knew of before 1898 occurred before he was born. He remembered seeing marks of that flood about knee high in the old barn. The 1898 flood was waist high, and some cattle had to be removed by swimming them out.

West Fork Little Pigeon River, mile 6.9—Charles Henderson, age 64 (in 1958), has spent his lifetime at his present home on the right bank of West Fork Little Pigeon River at mile 6.9, and has observed floods on the river since he was a boy. He showed an old cedar tree growing on a rock outcrop at mile 7.07. Notches on this tree, Mr. Henderson said, represent the crest levels of all major floods since 1908. Figure 55 is a photograph of the tree with the notches emphasized with white tape. The flood of April 1920 was the highest, the next was in 1928, the third highest was the flood of February 1957, the fourth occurred in 1908, and the bottom marks a flood in June 1932.

Approximation of destroyed marks—Marks such as those shown by figure 55 are among the most desirable forms of flood history data since the heights are not dependent on the memory of the witness. In some cases the marks even include the date of occurrence. Many floods, however, were not as carefully marked as these were, and of these a number have been carelessly destroyed. Time after time during the flood history work in the Tennessee Valley the investigators were told of marked trees that had been cut down or of marks on houses and barns that had not been transferred before the buildings were dismantled. In all such cases an effort was made, by questioning those who had seen the marks, to locate the tree or building site and to arrive at an approximation of the elevation of the mark.

RECORDED FLOOD DATA

The earliest official records of stream stages on the Tennessee River were initiated by U. S. Corps of Engineers during the period 1871 to 1875. The first official installation was a staff gage established at Florence, Alabama, November 6, 1871. This gage was observed by the Corps of Engineers until November 1, 1890, when the U. S. Signal Service took it over. In October 1894 the U.S. Geological

FIGURE 55.—*West Fork Little Pigeon River high water marks (emphasized with white tape) cut on cedar tree by lifetime resident of area.*

Survey began a discharge record at Florence which is continuous to the present day.

Other gages established on the Tennessee River by U. S. Corps of Engineers or U. S. Weather Bureau during the 1871-75 period were those at Paducah, Kentucky; Johnsonville, Chattanooga, Loudon, and Knoxville, Tennessee; and Decatur, Alabama. The Chattanooga discharge record is the oldest in the Valley, dating back to the installation of the original staff gage on April 1, 1874.

Before 1900, gages had been established also along the main river at Riverton, Alabama, at several locations in the Muscle Shoals area, at Guntersville and Bridgeport, Alabama, and near Rockwood, Tennessee.

On the tributary streams the first river stage record was obtained at a gage installed by U. S. Corps of Engineers on Clinch River at Kingston, Tennessee, in October 1874. After two years of observations this gage was abandoned until 1884 when the U. S. Weather Bureau took over the station. In February 1883 the Weather Bureau established a station on the Clinch River at Clinton, Tennessee, and another on the Hiwassee River at Charleston, Tennessee. These were followed by a gage on the Holston River at Strawberry Plains in 1885, one on the Powell River at Arthur, Tennessee, in 1892, and gages on Clinch River at Speers Ferry, Virginia, and French Broad River at Asheville, North Carolina, in 1895. In the lower end of the Valley a station was established on Duck River at Columbia, Tennessee, in January 1887.

In 1895 and 1896, the U. S. Geological Survey began discharge records on the tributary streams with stations on the French Broad River at Asheville, Hiwassee River at Murphy, Little Tennessee River at Judson, and Tuckasegee River at Bryson City, all in the North Carolina part of the Valley.

The gaging station network grew slowly in the Tennessee Valley during the next twenty years, with most of the new installations being on the tributary streams above Chattanooga. In September 1920, a three-party agreement was entered into by the Chief of Engineers, United States Army, the State Geologist of Tennessee, and the Director of the United States Geological Survey, for work in the Tennessee River Basin. The number of stations increased somewhat more rapidly at this time, but it was not until the 1930's that rapid expansion of the network began in the Valley. Since 1933 TVA has installed gaging stations for special uses and has been instrumental in the establishment of numerous gaging stations throughout the Valley through a cooperative agreement with the U. S. Geological Survey.

U. S. Weather Bureau data

Observations of river stages by the U. S. Weather Bureau are published in annual volumes of *Daily River Stages* which contain tabulations of once-daily readings at all Weather Bureau gages in the

United States. The first volume contained stages of the Ohio River and of its principal tributaries from 1858 to 1889. Beginning with the year 1907 special observations of high stages were footnoted. These footnotes were not always complete, however, and valuable flood history data were obtained by referring directly to the observers' original reports when these were available.

In many cases the observers made no special readings of the gage, and the regular 8:00 a.m. reading might be well below the actual crest stage. Where supporting evidence indicated that such was the case, and where the flood was of important magnitude, an attempt was made to reconstruct the stage hydrograph to arrive at an approximation of the crest. This reconstruction was always a doubtful procedure, especially on the smaller rivers, unless enough data were available at least to fix the time of beginning of the rise and the date and approximate time of crest. Where later discharge records at the site permitted development of unit graphs, this method was also used in estimating flood hydrographs and crest stages.

The U. S. Weather Bureau also has made available much valuable information of flood stages and storm rainfall in the publication, *Monthly Weather Review,* and the occasional supplements thereto. Examples of the latter are Supplement No. 29, "The Floods of 1927 in the Mississippi Basin," and Supplement No. 37, "The Ohio and Mississippi River Floods of January-February 1937."

It was important in using the U. S. Weather Bureau records to trace the history of each gaging station so that adjustments might be made for change of location or of datum. On the swift, steep streams feeding the upper Tennessee River a shift in location of a gage often makes a substantial difference in flood elevation.

U. S. Geological Survey data

Stage and discharge data collected by the U. S. Geological Survey are published in that Agency's annual bulletin, *Surface Water Supply of the United States.* These publications include, as part of the information for each station, a record of the maximum stage and discharge for the current water year and for the period of record. Additional detailed information on the stage and discharge of many streams during major floods is included in special reports on these floods. The more recent of these special reports also contain other pertinent hydrologic information and analyses and compilations of data relating to earlier noteworthy floods. An example is Water Supply Paper 1066, *Floods of August 1940 in the Southeastern States,* issued in 1949 and containing much information of value on the floods of August 1940 in the Tennessee Valley.

The installation of recording instruments by the U. S. Geological Survey did not begin in the Valley until about 1925 and 1926. Recording gages had been placed in operation as early as 1911 to 1914 by Tennessee Electric Power Company on Ocoee River and by Aluminum Company of America and its subsidiary, Knoxville Power Company, on Little Tennessee River, Tuckasegee River, and Nantahala River. Previous observations by the Geological Survey were once-daily or twice-daily readings of staff gages, with special readings during flood conditions. The twice-daily readings were usually averaged for publication. These staff gage readings were subject to the same limitations as those obtained by U. S. Weather Bureau observers and it was necessary in some cases to refer to the observers' original records and to reconstruct stage hydrographs in order to arrive at correct crest stages.

In the earlier Water Supply Papers, only the maximum stage for each water year is given which does not provide sufficient information for a complete record of the floods on a stream. In some years, three or four overbank floods may occur which would not be listed because they did not reach the height of the annual maximum flood. For this reason it was often necessary to refer to the original records and charts, when data on flood frequencies were needed, using the published data only to eliminate those years in which flood stages were below the minimum selected for study. Since 1947 the Water Supply Papers have listed data not only on the maximum flood for the year but also peak discharges for other important rises.

The U. S. Geological Survey records are the principal source of flood discharge data in the Tennessee Valley. The Water Supply Papers show for each water year the maximum flood discharge for the year, based on the rating curve then in use. In addition, the organization has on file original gage records and rating curves from which complete flood hydrographs can be plotted for determination of flood runoff.

At times, large floods have occurred that exceeded the range of discharge measurements at a station. Such floods were experienced, for example, in August 1940 in the eastern part of the Valley and in February 1948 in the western part. In these circumstances the Geological Survey has in recent years made a practice of obtaining post-flood discharge estimates near the station by the slope-area method, contracted opening method, or by measurement of flow over a dam. The peak discharge thus determined is used in extending the station rating curve. This procedure, while probably preferable to the extension of the rating curve on logarithmic paper or one of the other methods, has certain limitations which require that the results be used with caution. Peak discharges obtained from extended rating curves are compared and correlated with peak discharges at other stations on the stream or in the storm area to insure that the resulting flow is not inconsistent. Total runoff relations are similarly examined for consistency.

Improvement of rating curves as a result of recent high discharge measurements require a review of older records to determine whether peak discharges published for them should be revised to the new curve.

For the determination of peak discharge for pre-record floods, the latest rating curve for the station has been used, unless it was obvious that this curve represented markedly different conditions at the location. In view of the meager information available on possible changes of channel and control in the pre-record period, it was considered impractical to attempt any minor revisions of a rating in order to make it applicable to the early floods.

Other records

Much useful information was obtained from the files of the Aluminum Company of America covering streamflow and flood stages in the Little Tennessee River area since 1912. This company made intensive investigations of streamflow in connection with design of their dams in the watershed. Aluminum Company records of flood flows through Cheoah Reservoir were particularly helpful in studies of past floods at Fontana dam site.

Full use was made of the maps, cross sections, low water and streambed profiles, and high water marks collected by U. S. Corps of Engineers in their surveys of the Tennessee River and its tributaries.

CREST PROFILES

Profiles of the crests of past floods were prepared for each stream investigated and for every flood for which sufficient marks were avaliable. TVA has also made it a point to mark currently at least one important flood on the Tennessee River, on each major tributary, and in recent years on many smaller streams. These floods were marked on the crest in the case of the Tennessee River and as soon as was practicable after the crest on the tributaries. Marks were obtained at sufficiently frequent intervals to define the major breaks in the profiles, and these profiles have served as basic data for the development of the profiles of earlier floods.

The March 1936 flood was marked for this

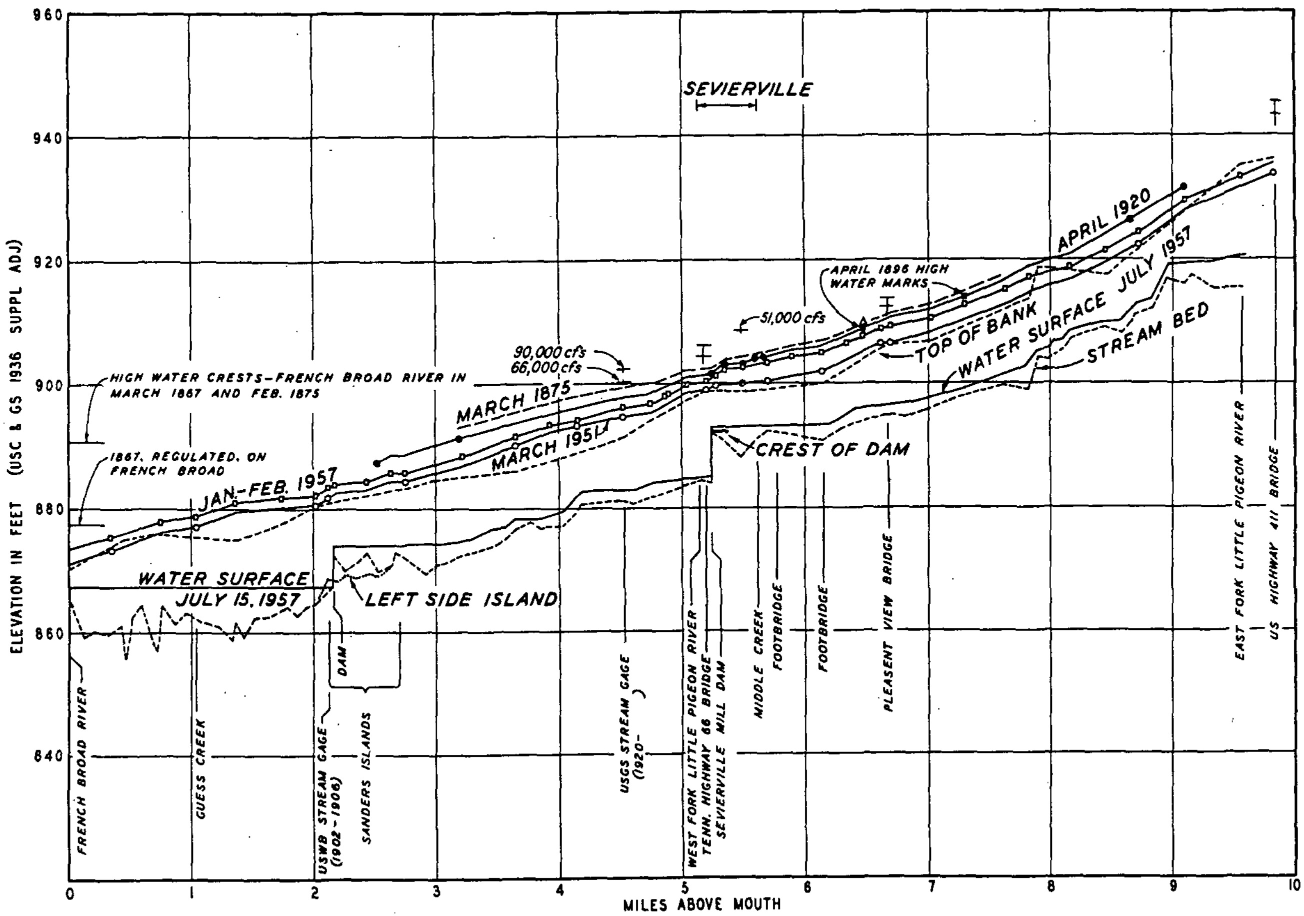


FIGURE 56.—*Segment of typical profiles of high water marks—urban area.*

purpose on the Tennessee River, covering the entire length from Knoxville to the mouth. On the tributary streams east of Chattanooga crest levels were marked for one or more of the floods of March 1935, January, February, March, and April 1936, August 1938, August 1940, March 1951, and January 1957. On tributaries west of Chattanooga, high water marks are available for floods of April 1936, January 1937, February 1939, February 1948, January 1949, March 1951, March 1952, and March 1955.

Before drawing profiles through the high water marks obtained from flood witnesses, a careful study of the description and history of each mark was made to evaluate its relative accuracy. Definite marks made at the time of the flood were given more weight than remembered marks, and marks based on remembered depths in or on buildings were given more weight than approximate positions of the water edge on the ground. Marks that plotted much too high or low were discarded after making certain that there were no errors in the levels.

In drawing the flood profiles, consideration was given to the shape of the basic profiles of a current flood and to the location of bridges, tributary streams and any other factors which might explain apparent inconsistencies in the marks.

Figures 56 and 57 are segments of typical profiles of high water marks. Figure 56 shows the results of intensive investigation in an urban area, while figure 57 covers a less populated reach along another stream.

Main river flood profiles and flood volumes and frequencies, and floods on the seven major tributaries are covered in detail in the next chapter.

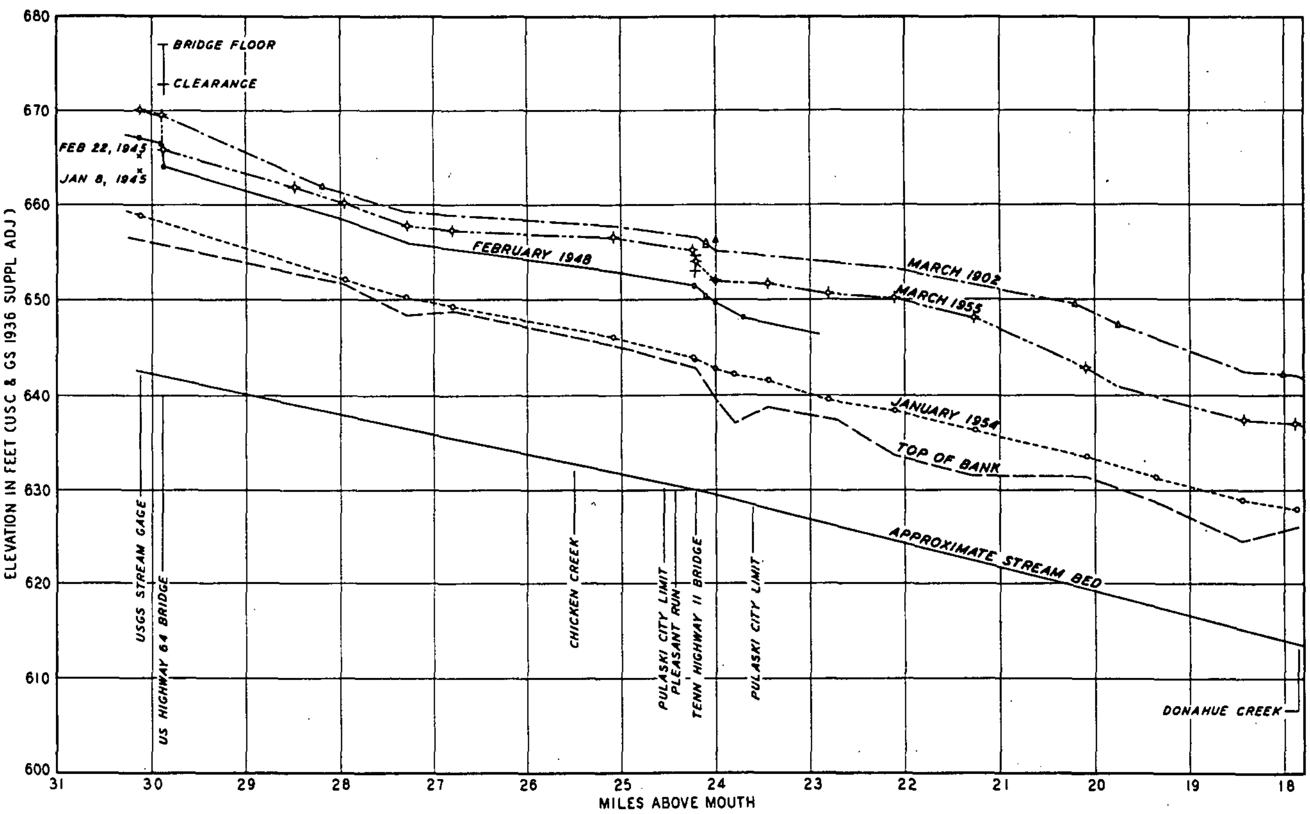


FIGURE 57.—*Segment of typical profiles of high water marks—sparsely populated reach.*

CHAPTER 4

TENNESSEE BASIN FLOODS

Floods in the Tennessee Basin—as presented in this chapter—are divided into two categories: main river and tributary. With respect to the main river the chapter tells about the preparation of flood profiles and discusses flood volumes and frequencies. Concerning floods in the tributary basins, the chapter describes each major tributary area and the floods that have occurred therein, and gives tabulated data on maximum known floods in each basin.

MAJOR FLOOD PROFILES TENNESSEE RIVER

Profiles of crest water surface elevations in past floods on streams within the Tennessee River Basin have been given a considerable amount of attention and study, and the concentrated and thorough study of the flood history and physical characteristics of the main Tennessee River resulted in profiles that have served many uses, some of which were direct, and others, none the less important, were indirect.

Profiles of past floods, aside from their historical value, are indicative of what may be expected in the future. They furnish basic elevations for the determination of areas which have been flooded in the past, and permit a comparison of past flood elevations with estimated future elevations resulting from changes or artificial obstructions in the river channel. Such a comparison in turn provides means of assigning flood benefits or additional flood responsibility, as the case may be.

Profiles of floods have also served widely as elevation criteria for bridge and transmission line clearances, river terminals, industrial plants, and other developments and structures along the river. Of more indirect nature, although closely related to most of the above uses, flood profiles have been used extensively to provide basic data for the preparation and checking of computed water surface profiles, for the extension of stream gage relationship and rating curves, and for similar or related hydraulic studies.

The earliest profiles made use of flood mark data then available from field work by the U. S. Corps of Engineers conducted throughout a long span of years, ending when TVA was created. It was soon recognized, however, that additional field data were needed. Reservoir survey parties were alerted to watch for and survey flood marks which were encountered in the course of their other activities. Results of this incidental search, however, were inadequate. So as to provide the kind of data most needed, and to insure that search would be made where data were most needed, a field party was assigned the specific task of locating and surveying highwater marks. By a thorough and careful search this party was able to locate and survey more than 200 valuable marks never previously surveyed. Many other marks were revisited and their accuracy improved. By means of close liaison between the field party and the office, uncertainties and confusions in marks and profiles were effectively resolved.

Data available

Records of the heights to which past floods on the Tennessee River have risen have been preserved in several ways. River gaging station records usually are considered to be the most reliable. Yet gage records frequently are deficient either in length of record, in accuracy of zero base level, or completeness and accuracy of flood stage readings. Changes in zero elevation or gage location frequently pose difficult problems. At best, there were never enough gages along the Tennessee River in the past to define flood profiles in more than a most general way. Nevertheless, data from all available gages have been used, but only after a thorough historical investigation of the station to establish the location and zero level appropriate to the period of record being used.

The most useful and reliable flood height data have been the rather abundant flood marks that were located by field survey. Even as late as 1939 a large number of clearly defined and labeled marks on relatively permanent objects were found for floods as ancient as 1826. Also, many actual witnesses were found to floods as early as 1867. Informed descendants of witnesses were more numerous. Old-time residents along the river were generally hospitable and cooperative with the field party, even though occasionally they were overzealous to the extent of enlarging upon limited knowledge of the event to the detriment of accuracy. Flood marks located by field survey vary from prominent carvings on trees and buildings showing the initials "HWM" and the date of occurrence to memory positions on sloping ground.

Data are adequate to define profiles throughout the entire length of the Tennessee River for the two great historic floods of March 1867 and March 1875. The comparatively recent flood of March-April 1936, except in Wilson Reservoir, was field-marked from a boat at close intervals by TVA engineers as the flood

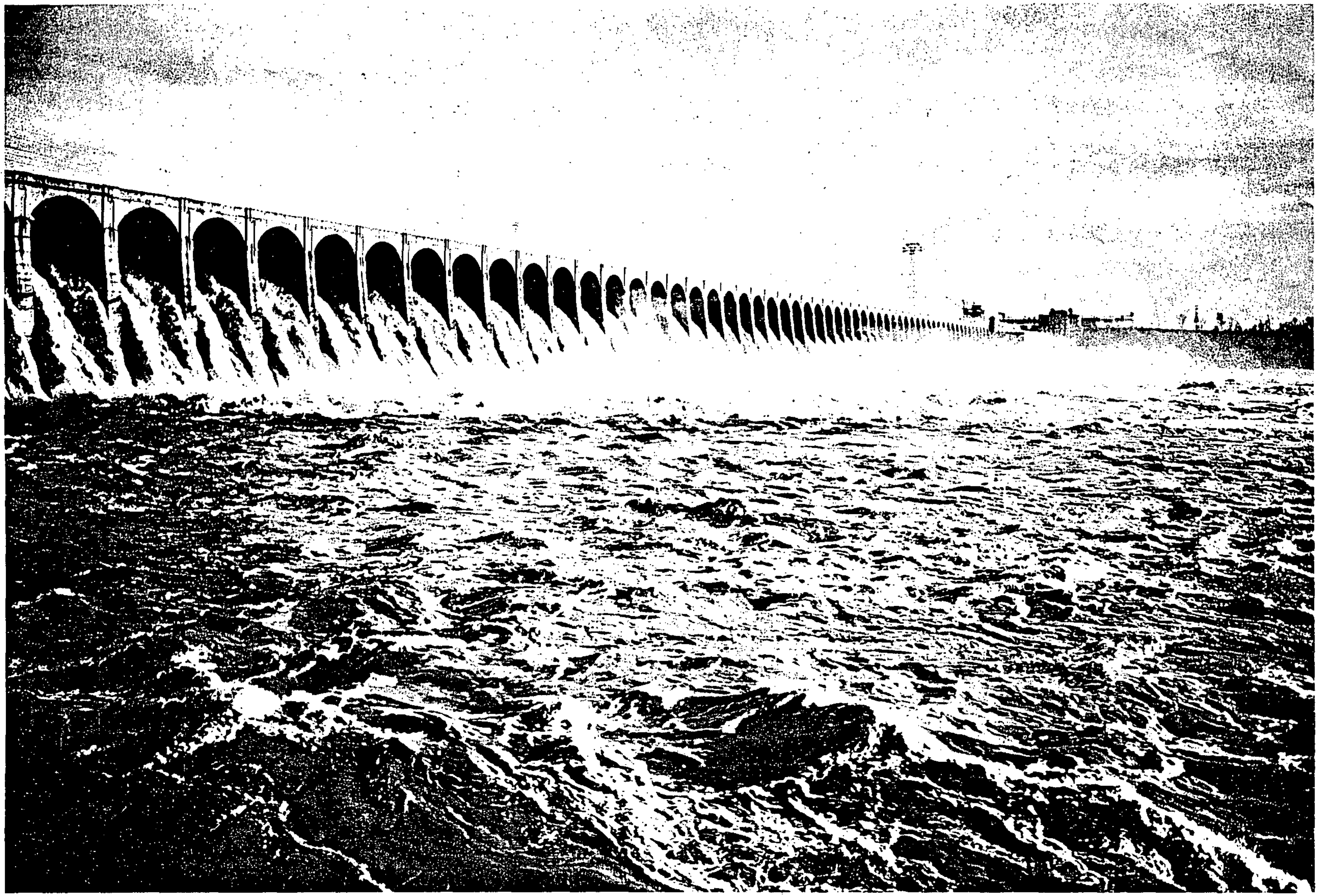


FIGURE 58.—*Wilson Dam discharging flood waters in 1936 (this was before completion of Norris Dam, the first TVA-built project).*

was in progress. The 1936 marks were carried through Wheeler and Hales Bar Reservoirs, which were in operation at the time. Marks for this flood were therefore more dependable than for any previous flood studied.

Data also permitted preparation of profiles over substantial sections of the Tennessee River for the floods of January 1882, April 1886, March 1897, March 1917, December-January 1926-27, and January 1937.

Most of the marks used to define historic floods on the Tennessee River below Chattanooga were originally located by the U. S. Corps of Engineers in surveys dating back as early as 1875. Early surveys enjoyed advantages over recent surveys, but suffered certain compensating deficiencies. Living witnesses were more numerous and their memories were fresher. Flood marks also were fresher and more plentiful. Survey methods and datum networks, however, were far less accurate and complete than in recent years. Elevation data in particular were different and confusing. Through careful analysis of original field notes it has been possible to circumvent many of the disadvantages of early surveys and yet retain their advantages. By means of references in the original surveys, and sometimes with minor supplemental surveys, a large number of valuable marks were salvaged by referring them to present-day datum and river mile.

On the Tennessee River above Chattanooga similar techniques were employed, but most reliance was placed on the marks located by the field party specifically assigned to that task. More than 190 new marks were located in this field work above Chattanooga and about 50 new marks below Chattanooga. In both sections of the river a number of good flood marks were lost by close time margins due to reservoir clearing. Other marks were saved from destruction by equally close time margins. Marks were also barely saved and others barely lost by the continued survival, or the unfortunate death, of aged flood witnesses or informants.

Accuracy of data

Viewed in its entirety, the flood-mark data for the Tennessee River are relatively accurate. The marks for the 1936 flood, having been made from a boat as the flood was in progress, are especially dependable. Some marks for historic floods probably are equally dependable, but most of them are not. No completely satisfactory technique has been devised for separating historic flood marks into "reliable" or "uncertain" categories.

The profile in figure 59 uses a separate distinguishing symbol for each major flood but shows it either solid or open in accordance with the following notation which appears on the profile:

> Solid symbols represent flood marks whose locations on well defined objects have been established from available records and whose elevations have been determined by spirit leveling. Open symbols represent flood marks whose histories, descriptions, or lack thereof, indicate that they may be uncertain in location, elevation, or both.

Several influences may introduce error into flood marks which appear to be reliable. The elevations of marks on buildings may change due to building settlement, reconstruction, moving, or through inaccurate transfer of the mark from one place to another. Unrelated carvings on trees can become confused with flood marks, and ancient carvings may not be legible. The effect of curvature of the stream and the changes in velocity head of flowing water also can introduce apparent inconsistency in the profile even though a mark is carefully made.

Generally speaking, the marks for the maximum known flood, even though it occurred many years ago, are more abundant and usually more dependable than marks for lower, though more recent, floods.

The absence of complete information regarding flood marks, except for those collected by TVA, was a major handicap in judging the comparative reliability of conflicting marks. So far as practical, this deficiency was rectified by revisiting the marks and the informants. In an effort to prevent similar deficiencies in later flood-mark surveys, a field report form was prepared stipulating the information required and providing a space for its entry. A copy of this form is shown in figure 60.

Preparation of profiles

The initial location of profiles with respect to the available flood marks was made on trial work sheets, showing also the low-water profile by the U. S. Corps of Engineers. There was little choice of position for the well-marked but relatively low 1936 flood, and this flood then became a plane of reference for other floods. The generally greater abundance of marks for the maximum known flood fixed the tentative position of the 1867 flood from the head of the river downstream to the mouth of Elk River, and the 1897 flood below this point. Profiles for other floods, of intermediate height were then prepared, using appropriate marks, and with the 1936, 1867, and 1897 flood profiles as general guides.

Frequently marks for several floods exist on the same object, such as a tree, porch column, or house corner. Relative to each other the floods compare favorably with other relative data, yet elevations sometimes are difficult to reconcile. Similar relative heights without known elevations may appear in family records, diaries, historical accounts, and newspapers. Adjustments in the tentative profile positions were made to conform with such relative data. The relationship was particularly evident between the 1867 and 1875 floods along most of the river above Chattanooga. This established relationship often permitted more accurate location of the 1875 flood profile from 1867 flood marks than from 1875 marks

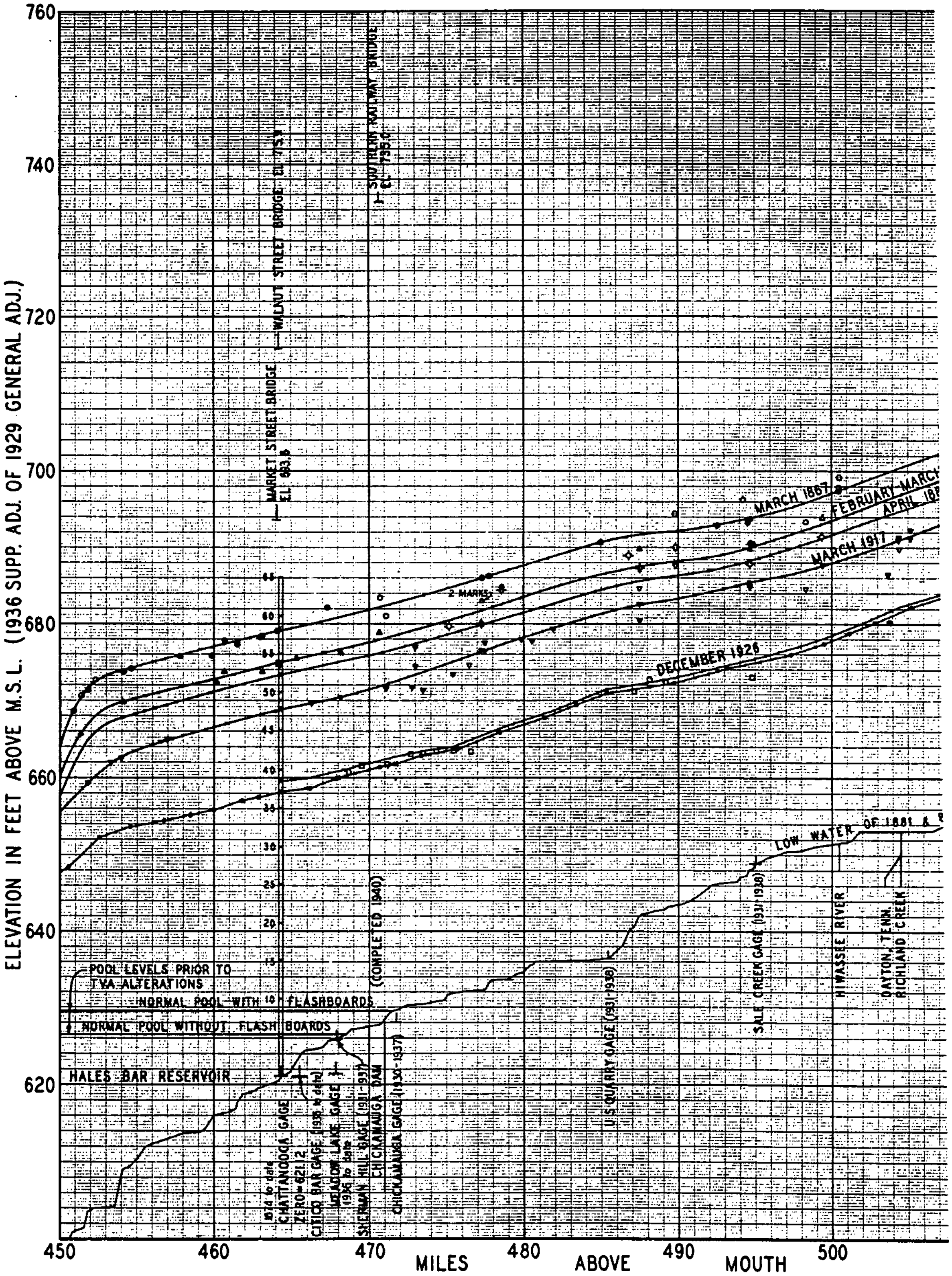
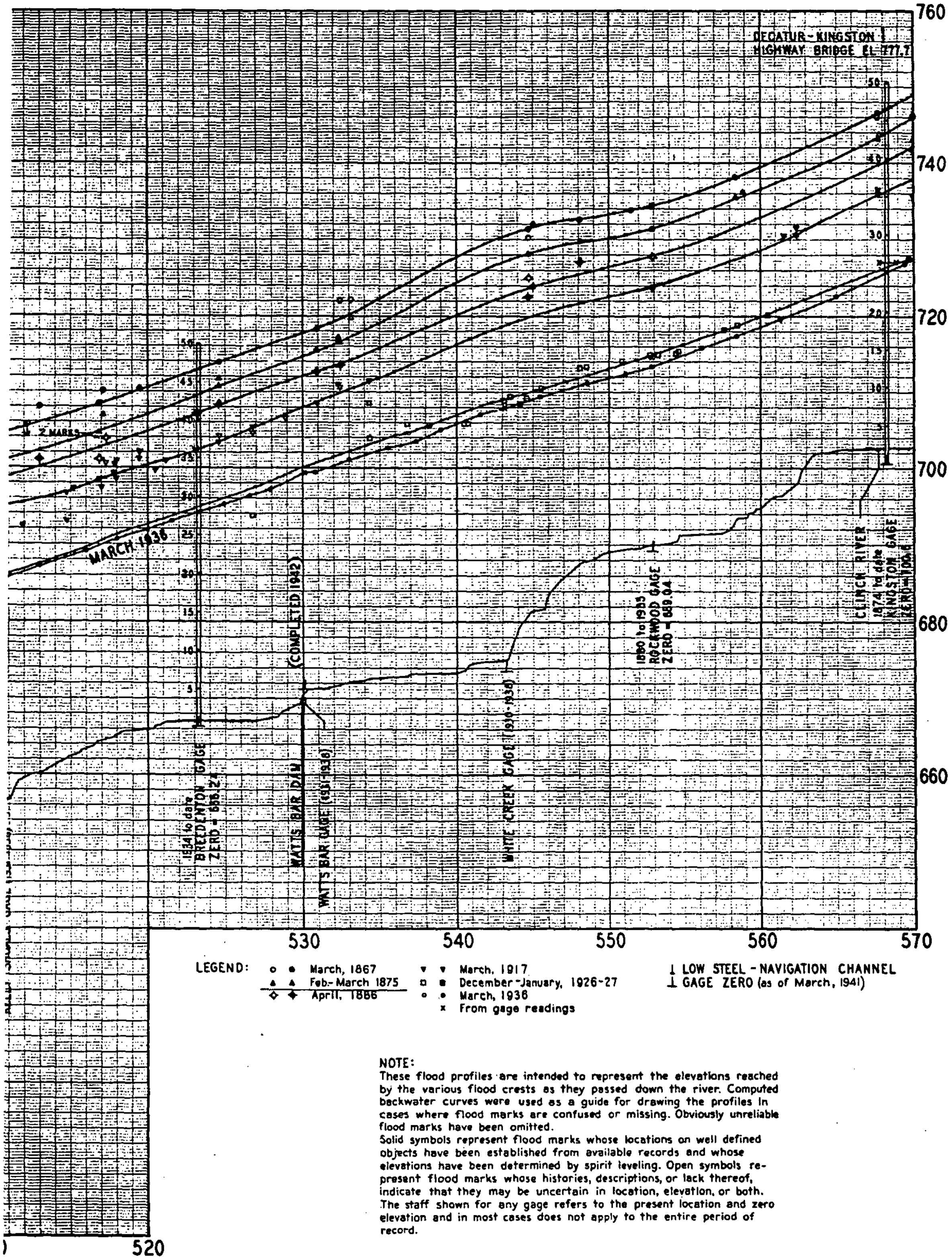


FIGURE 59—*Flood profiles—Tennes:*



er from mile 450 to mile 570.

TVA 4045
Tennessee Valley Authority
Flood Control Section

Date of Flood ________
River Mile ________
Elevation ________

REPORT ON FLOOD MARK

High Water Mark ____________; River ____________; River Bank ________

LOCATION AND DESCRIPTION OF MARK

State ____________ County ____________ Nearest town or city ________
Quadrangle ____________ Approx. Lat. ________; Long. ________
Located on land of ________________________
How to get to vicinity of mark ________________________

Description and reference to conspicuous land marks ________________________

History of mark ________________________

Distance from river ________________________
Where on river the determination is effective ________________________
Assumptions made in assigning elevation to mark ________________________

Elevations obtained from ________________________
________________________ Date ________
Levels in field book ________ Page ________ Party ________
Conclusions on reliability of determination ________________________

REPORT OF INTERVIEWS

Interview with ________________________ Date ________
Address ________________________
Interviewed by ________________________
Brief of interview ________________________

FIGURE 60.—*Field report form used for recording flood mark data.*

alone. It also served to "firm-up" the 1867 profile position in some places where only 1875 elevations were known.

After the profiles had been located as well as possible from the flood marks, further improvements were made by comparison with computed water surface profiles. Use was made of computed profiles in a few locations at which the computed profiles and the then available marks could not be reconciled, despite efforts to make all reasonable modifications to the computed profiles. In these cases, additional field surveys located data which supported the computed profiles. Thereafter, computed profiles were used to clarify confusing flood-mark data and in reaches where marks were missing.

Profile drawing

Final preparation of the main Tennessee River flood profiles followed the consideration of all available field and gage data, and of specially computed water surface profiles. Profiles for several major historical floods of the Tennessee River between miles 450 and 570 are shown in figure 59. This figure is one of a series of drawings that show the three or four floods known generally to be the highest on the stream, although the relative positions of some of the floods interchange over the length of the river. In fact, a major important flood on one part of the river might be unimportant and not even show in another part of the river. An example is the 1897 flood which, though the maximum known on many miles of the lower river, is not important enough to show above the mouth of the Elk River.

In addition to showing the highest three or four flood profiles at all points along the Tennessee River, profiles of other lower and well-marked floods of general interest and usefulness are given. All available flood marks are shown on the profiles. There is no question but that the highest profile of human knowledge extending back to 1826 is represented on the drawings.

Figure 59 also shows the low-water profile, the location and zero elevation of river gages used generally prior to the inception of TVA, important tributary junctions, important towns, the sites of TVA dams, and the location and low steel of the then existing bridges.

In addition to profiles showing elevations of specific floods it is frequently desirable to know the relation between elevation and discharge at any point along the river, not only where stream gages have been established and rated. Accordingly, flow profiles were computed by backwater methods for river conditions existing before TVA. These profiles for the Tennessee River from mile 430 to mile 652 are shown in figure 61. The approximate discharge of a flood shown in figure 59 may be obtained by applying the elevation of that flood to the flow profiles, figure 61. The flow profiles also present sufficient information for plotting a reliable rating curve at any point along the river, as at a dam or powerhouse site.

Peak stages and discharges of maximum known floods at five main Tennessee River gaging stations are tabulated in chapter 5 (table 15, page 109) together with data for tributary stations. The heights of floods at Chattanooga on the Tennessee River are shown graphically in figure 25, page 22.

FLOOD VOLUME—TENNESSEE RIVER

Volumes of runoff in ten major floods at four Tennessee River gaging stations (Knoxville, Chattanooga, Florence, and Johnsonville) are given in table 6. Information for the 1867 flood, the greatest known on the upper river, is not available. The volumes, given in units of day-second-feet (1 day-second-foot = 2 acre-feet approximately) and inches over the drain-

TABLE 6.—*Flood volume and peak discharge—10 major floods at 4 Tennessee River gaging stations.*

Date of flood	Tennessee River: Peak, cubic feet per second	Volume: Day-second-feet	Volume: Inches
Knoxville (Drainage area=8,913 square miles)			
February 1875	270,000	1,058,900	4.41
April 1896	177,000	676,000	2.82
February 1897	137,000	567,000	2.36
May 1901	186,000	804,400	3.35
December 1901	169,000	988,600	4.11
March 1902	197,000	1,014,100	4.24
March 1903	118,000	534,300	2.22
July 1916	175,000	720,300	3.00
March 1917	170,000	906,600	3.78
April 1920	149,000	648,800	2.70
Chattanooga (Drainage area=21,400 square miles)			
March 1875	410,000	4,734,300	8.23
March 1884	297,000	3,092,400	5.37
April 1886	391,000	4,525,600	7.86
March 1890	294,000	2,610,100	4.54
March 1891	264,000	2,631,800	4.57
April 1896	276,000	1,855,100	3.22
January 1902	279,000	2,457,100	4.27
March 1917	341,000	2,780,000	4.84
February 1918	289,000	2,148,400	3.74
April 1920	298,000	2,653,800	4.60
Florence (Drainage area=30,810 square miles)			
March 1875	400,000	8,233,000	9.95
March 1880	313,000	4,456,000	5.39
February 1884	307,000	6,211,000	7.51
March 1884	325,000	4,985,000	6.03
April 1886	380,000	6,263,000	7.58
April 1892	304,000	3,470,000	4.20
March 1897	470,000	6,655,000	8.05
March 1899	326,000	4,976,000	6.02
March 1917	317,000	4,500,000	5.45
December 1926	350,000	4,329,000	5.24
Johnsonville (Drainage area=38,530 square miles)			
March 1880	329,000	7,493,500	7.25
April 1886	372,000	8,461,800	8.19
March 1897	475,000	15,297,300	14.80
March 1899	360,000	8,079,000	7.81
April 1902	331,000	4,210,300	4.07
April 1911	313,000	6,453,500	6.25
April 1920	318,000	5,655,300	5.46
March 1922	318,000	7,786,000	7.53
January 1927	367,000	6,306,500	6.10
March 1927	319,000	4,382,000	4.24

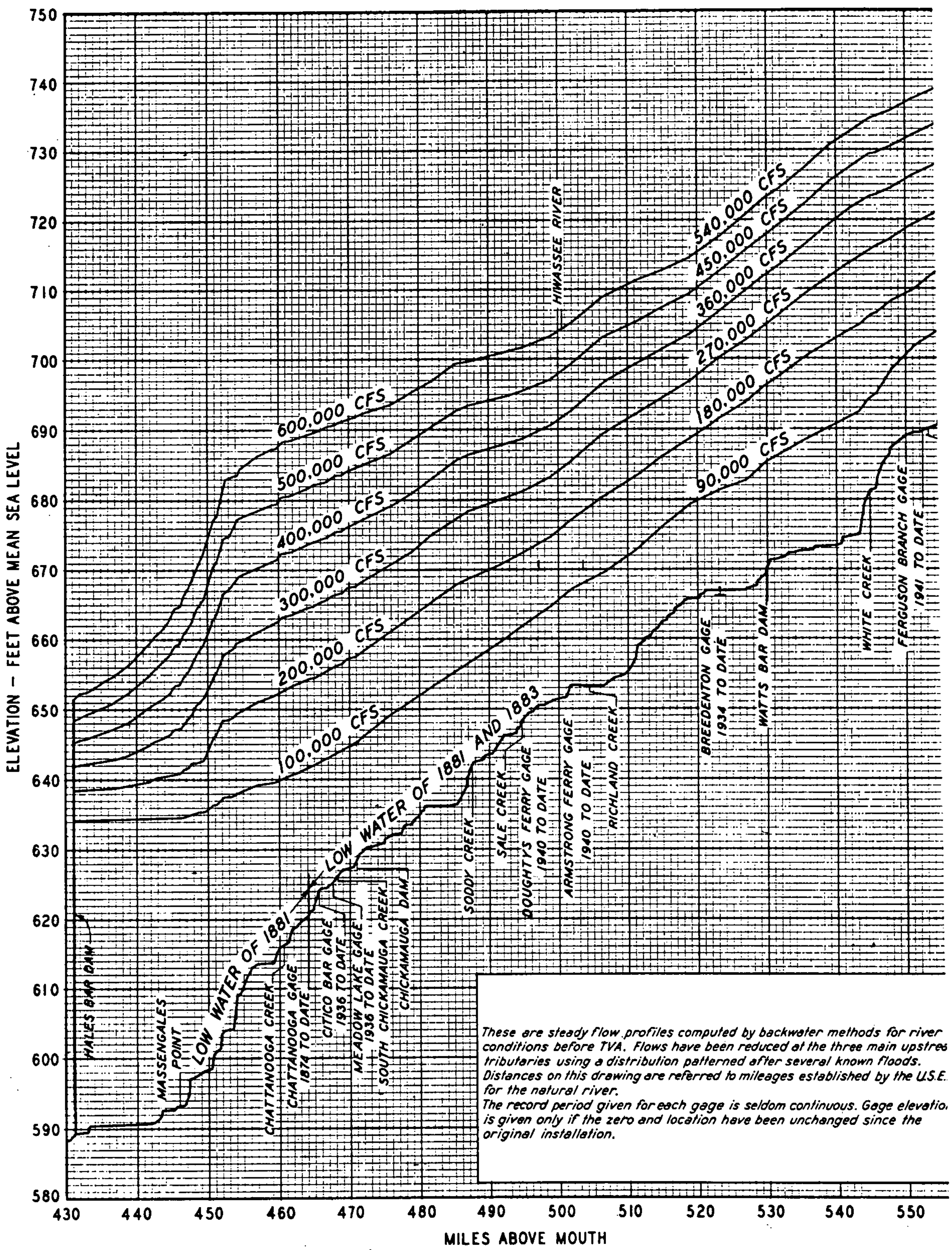
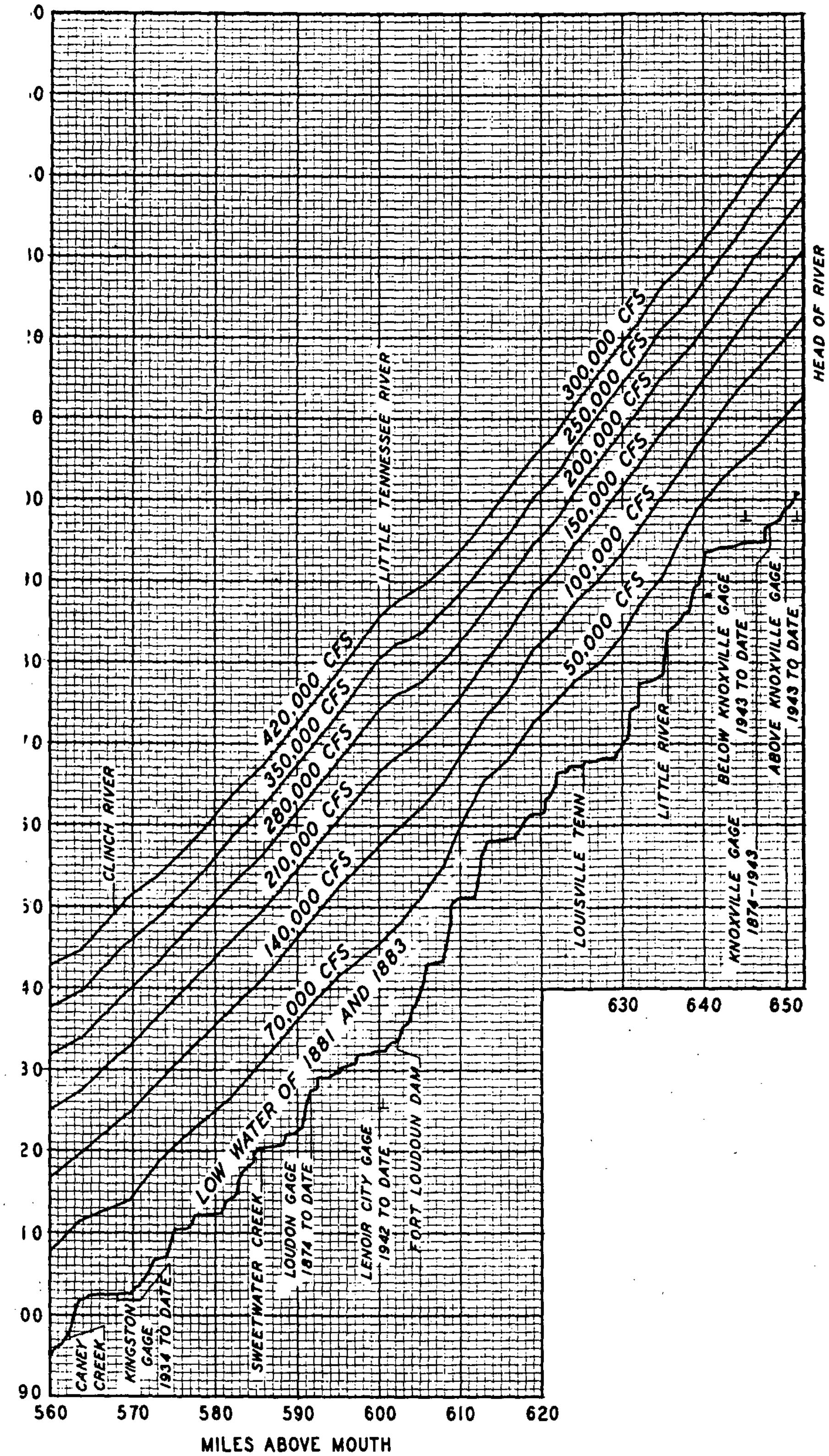


FIGURE 61.—*Flow profiles for Tennessee R*



litions before TVA—mile 430 to mile 652.

age area, include the entire flow without correction for base flow or runoff from prior rainfall.

The volumes of course are greater as the size of the drainage areas increase, not only in terms of day-second-feet but also in terms of inches over the drainage area. This reflects the greater duration for the large areas and the possible occurrence of more than one storm within the flood period. Although there is an approximate direct relationship at each station between the peak discharge and the flood volume, it is not defined well enough to show in diagrammatic form.

FREQUENCY OF TENNESSEE RIVER FLOODS

The term "flood frequency" is used here as the average interval between flood occurrences, with consideration being given to the inclusion of historic floods which occurred before stream gages were installed. Flood frequencies are useful in establishing heights of cofferdams at construction projects, elevations of navigation facilities such as lock walls, and elevations of various river crossings such as bridges and transmission lines. They are also useful, in conjunction with a flood damage curve, for the determination of average annual flood damages.

The flood distribution chart, figure 25, page 22, shows the chronological and seasonal occurrence of floods at Chattanooga. This chart shows indirectly the frequency of various flood heights, but to determine the frequency in terms either of average number of floods per year or of average interval between floods it is necessary to keep the length of flood record in mind.

Data available

Flood data for use in the determination of frequency comes from formal records at gaging stations, from historical records, and from results of routing computations. Long-term stage records are available for gaging stations along the Tennessee River at Knoxville, Loudon, Chattanooga, Decatur, Florence, and Johnsonville. Florence and Chattanooga records are continuous since 1871 and 1874, respectively. At the remaining stations records were begun in 1874 or 1875, but readings are not continuous. At Decatur, where gage readings were begun in 1875, only minor gaps exist in the record, and readings are continuous since in 1895. At Johnsonville, gage readings are continuous since 1885, two earlier years are without record, but gaps in other years are not extensive. At Knoxville and Loudon no gage readings were made from 1877 through 1882. Thereafter, at least winter readings are available at Loudon until 1898 and at Knoxville until records became continuous in 1899. Loudon records became continuous in 1904. The readings for these gaging stations constitute the formal records upon which much of the frequency study was based.

The study to determine flood frequency was made in terms of flow so that neighboring stations could be compared, and also because flow has greater hydrologic significance than stage. This is especially true now that the reservoirs have been completed. Flows for long periods in the records of all six gaging stations have been published by the U. S. Geological Survey using such ratings as were currently available at the time. Some large floods which occurred before ratings were well established are included in these periods. In most cases the entire station history at a gage, and extensive backwater computations, were used to establish good ratings.

The highest known Tennessee River flood from the head of the stream down to about the mouth of the Elk River was that of March 1867. Below this it was exceeded by the March 1897 flood. Ohio River backwater in 1937 caused higher elevations than in 1897 on the lower 47 miles, but the Tennessee River flow was too low to have much effect on the frequency study.

The earliest known floods on the Tennessee River were in 1791 and in 1826. The 1826 flood is said to have been about 8 feet or more below the 1867 level at Guntersville, mile 358. At the same place a flood in 1847 was reported to have been about the same height as in 1826. A Judge Galbraith is supposed to have cut the 1847 mark and said that the 1826 high water was the same. At Chattanooga, about mile 464, the 1826 flood is said to have been within a foot of the 1847 flood. According to the *American Union* of March 14, 1867, the 1847 flood at Chattanooga was 15.5 feet below the 1867 flood. Farther upstream, at mile 544.7, the 1847 flood was about 12 or 13 feet below the 1867 flood. There is no further mention of the 1826 flood.

Routing computations gave natural crests for floods which have occurred since construction of TVA projects. These routed crests were used in the natural frequency computations just as though the regulating dams had not been built. Routing computations were also used to get regulated crests for historic floods.

Frequencies of low floods

The average intervals between quite low flood crests have been computed entirely from the formal gage readings simply by dividing the period of record by the rank or position of each flood in the total array of floods. This assumes, correctly, that low-ranking floods have been equalled or exceeded a representative number of times during the period of record. The correctness can be checked by first separating the total period of gage records into two or more parts, equal in length or otherwise. It will be found that the one-year flood determined from each part will be nearly the same as the one-year flood determined from the entire record.

If there is a fairly long record, the flood rate which has been equalled or exceeded as many times as there are years in the record can be considered a one-year flood. Also, a rate which has been equalled or exceeded a number of times equal to one-half the number of years in the record is a two-year flood. Frequencies computed by division in this way plotted smoothly from one-year occurrences, the lowest determined, upward to five- and ten-year occurrences or even higher in some cases.

In computing the period of record, fractional parts of a year were added for years in which the records were not continuous. The size of the fraction depended partly on the season. On the Tennessee River most floods occur in the winter months, so a station having readings from December to March, as often was the case, was credited with an effective three-fourths of a year for each such one-third actual year of record. Summer and fall months, if given alone, were credited as less than their actual proportion of a year because these are dry months.

A record period for a gage could sometimes be extended by comparison with neighboring gages having continuous records. For example, if Knoxville showed no flood in a year of missing or partial Loudon records, that entire year was counted in computing the total Loudon record period. This assumed that no flood occurred at nearby Loudon if none occurred at Knoxville.

Comparison with nearby gages was used even during flood years. For missing Loudon years during which floods occurred at Knoxville, it was logical to assume that floods of the same relative size occurred at Loudon. Without assigning a crest at Loudon to these floods, a position of rank was assigned, regardless. This left a space for the flood in the total array of all floods. Then such missing years became a part of the total Loudon period used for computing frequencies for all other floods. The missing floods did not become plotting points, but had an influence on the plotting position of other floods.

Frequencies of high floods

Plotted frequencies computed by the simple division of the rank into the period of record scatter much more for high floods than for low ones. Such scattering is to be expected because the higher floods, being fewer in number, are not as representative of long-term average conditions. This can be proven easiest by dividing a given set of records into two or more equal parts. Then the maximum flood in each part will have equal frequency, yet the flood crests almost never are equal. Nor will the second, third, or fourth high floods in one part of the record usually equal their counterparts in another part of the record.

The entire formal record is in itself only a part of the total past history, known or otherwise. Therefore, the few high floods of the entire record may not be completely representative of long-time performance. This suggests that a variation in procedure which in effect extends the period of record would make the few higher floods more representative of long-term performance. Such a variation is the inclusion of a period of historic flood knowledge along with the formal records. If the entire river is treated as a unit, this addition to the record is more effective and more correct than if gage locations are treated separately. Extending the effective record period back in time by adding historic floods is based on certain logical assumptions.

The preservation in the region's history of knowledge about floods in 1826 and 1847 is taken as proof that higher floods did not occur until the great flood of 1867. Otherwise, there would also be knowledge of such floods. Therefore, the 1867 flood was the greatest in that early 42 years. In the short period from 1867 to 1872, when the earliest Tennessee River gage record was commenced, no important floods are a part of the region's history. This adds five more years to the effective period. Several gages were in operation in 1875 when a flood occurred which was second only to that of 1867 from the head of the river also down to the Elk River. Some of the early gages became inactive after 1876 until about 1883, but enough remained and enough flood marks are available to show that the 1886 flood was next in magnitude below that of 1875 in the same length of the river. The 1867, 1875, and 1886 floods all were clearly above the 1826 and 1847 floods.

Thus, frequencies for these three floods above the Elk River can be computed using an effective record period beginning with 1826. Below Wheeler Dam site the 1897 flood may be treated in a similar way, as may also the 1882 flood on parts of the stream where it becomes second only to the 1897 flood.

It is comparatively simple to show the correctness of an effective period of record extending back through 1826 when computing frequencies for the few known highest floods. These floods may be either within or before the formal gage records. It is not possible, however, to know just where the use of this longer effective period of record should end when frequencies for lower floods are computed. In actual practice good results were obtained by first consciously trying the expanded effective period on lower floods than seemed proper. Next, a shorter period (perhaps that of the formal gage readings) was tried on floods higher than seemed proper. A logical separation point appeared when the overlapping results were plotted.

Logic would defend extending the effective record period even back beyond 1826. For example, it is not likely that a flood in 1825 equalled or exceeded that of 1826. This would add a year to the useful record. The same could be said also of 1824, which would add another year, and for 1823, and so on. A period back beyond 1826 could be used safely to compute frequencies for some of the highest floods. Unfortunately, this logic has no well-defined stopping point. Perhaps the date of earliest settlement in the area would serve. No effort has been made in the present study to so extend the effective record period, but in preparing the drawings for some locations the curves were located somewhat beyond the computed frequency of the higher floods. In effect, this accomplishes the same purpose.

Adjustment of frequency

At first only tentative curves were drawn through plotted values. Next, an adjustment curve was made by which the gage curves, all for the Tennessee River, could be brought into relative agreement. This was done by plotting discharge from each tentative gage curve against drainage area for several chosen frequencies. Lines connecting points of equal frequency then became the tentative relationship of flow against drainage area for the Tennessee River. Smoothing was done with the curves in this form, taking into account the known character of the stream.

No smoothing of the one-year frequency line was necessary, presumably because of all frequencies the one-year flow is most accurately established by observed data. Smoothing of the frequency lines for higher flows, therefore, was guided largely by the one-year flow.

Data from this smoothing curve next were transferred back to the individual gage curves fixing the final positions for each. Tentative curves did not have to be changed much in this adjustment.

Natural flood frequency curves

Figure 62 shows the natural flood frequency curve for the Tennessee River gaging station at Chattanooga. Similar curves were drawn for other river gages. The adjustment curve used in smoothing the frequency curve at gaging stations was also used to determine frequency curves for each dam site on the Tennessee River.

Regulated flood frequencies

The period since the TVA reservoir system has been mostly completed is too short, at the most only since 1936, to include enough observed regulated floods to establish a regulated frequency curve. These data can be supplemented, however, by regulated flows that have been computed for a number of historic floods, using operating procedures which give results about comparable with actual regulation. Plotted points showing regulated frequencies did not indicate a definite curve position using the data available at the time of plotting.

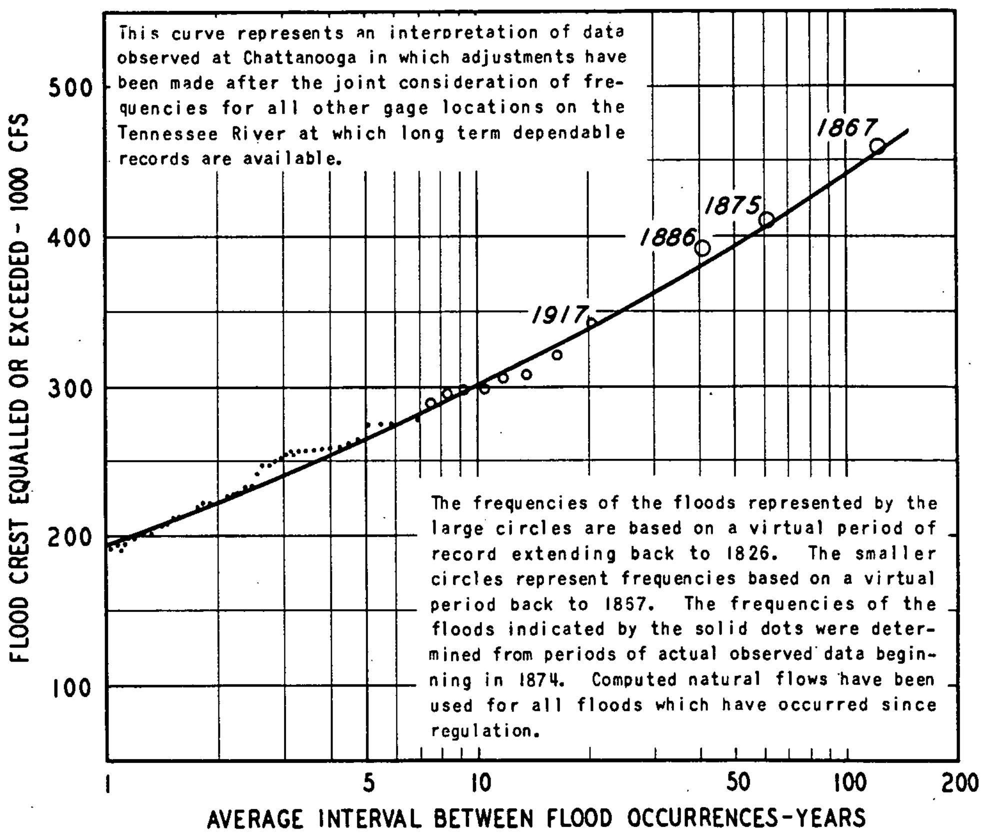


FIGURE 62.—*Natural flood frequency—Tennessee River at Chattanooga, Tenn.*

FLOODS—MAJOR TRIBUTARY BASINS

The Tennessee River has seven major tributaries, two entering the river west of Chattanooga, mile 464, and five entering east of that city. Crest profiles of past floods on these tributaries are discussed in chapter 3. The seven tributaries with their point of entry and drainage area are listed in table 7, and figure 63 shows their locations in the Valley.

TABLE 7.—*Major tributaries of Tennessee River.*

Tributary	State of origin	River mile at confluence	Drainage area, square miles
Duck River	Tennessee	110.7	3,500
Elk River	Tennessee	284.3	2,249
Hiwassee River	Georgia	499.4	2,700
Clinch River	Virginia	567.7	4,413
Little Tennessee River	North Carolina	601.1	2,627
Holston River	Virginia	652.1	3,776
French Broad River	North Carolina	652.1	5,124
Total major tributary drainage			24,389

The major tributaries drain some 60 percent of the Tennessee River Basin and the drainage areas of these tributaries, together with floods that have occurred therein, are discussed in the following paragraphs.

The minor tributaries all have drainage basins of less than 1,000 square miles. The largest is Bear Creek with 946 square miles of watershed in northern Alabama and Mississippi.

Duck River Basin floods

The Duck River, the largest tributary of the lower Tennessee River, has its origin on the eastern Highland Rim of the Cumberland Plateau in Coffee County, Tennessee. Figure 64 is a map of the watershed. From the relatively mountainous area in the headwaters the river flows westward across the rolling terrain of the Nashville Basin through one of the richest agricultural regions in Tennessee. The lower 50 miles of Duck River and the entire watershed of its principal tributary, Buffalo River, lie in the western Highland Rim area. Buffalo River drains an area of 764 square miles while Piney River, another tributary, has a drainage area of 223 square miles.

The Duck River Basin, which is 53 percent forested, lies approximately normal to the path of heavy winter storms moving northeastward across the Tennessee Valley and, because of the great length of the area, the heavy rainfall is rarely distributed uniformly over it. These variations in rainfall distribution, coupled with changes in the character of the stream itself, result in differences in relative magnitude of floods in the upper and lower reaches of the main stream and on the tributaries.

The four principal towns of Manchester, Shelbyville, Columbia, and Centerville are all located on the river and partially on the flood plain. Wide areas of farm land are subject to overflow, and numerous bridges and roads are vulnerable to flood damage.

The only regulation on Duck and Buffalo Rivers is by small power plants and mill dams, none of which have any effect on flood flows.

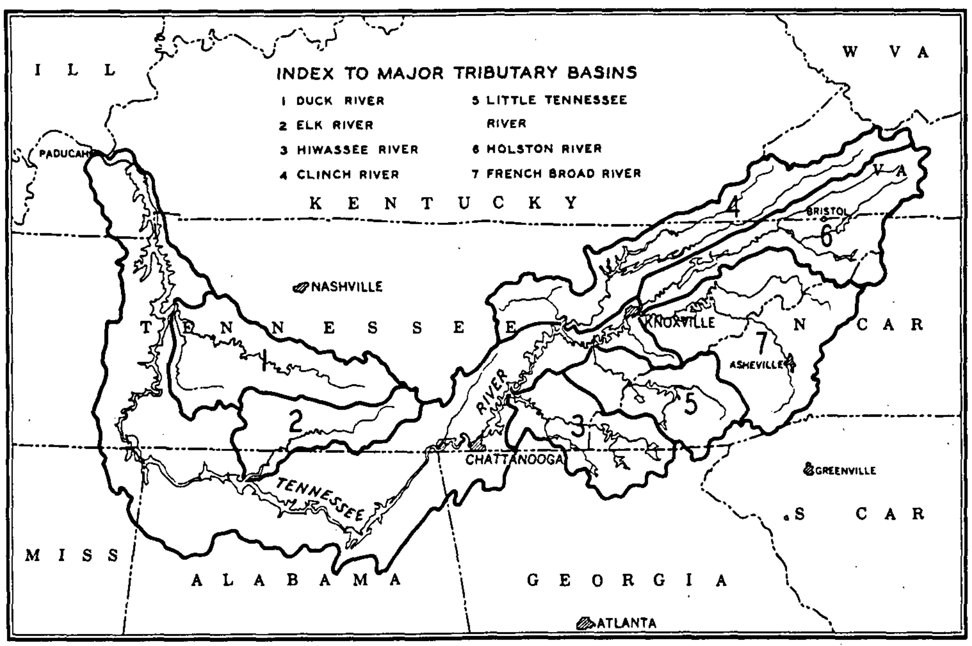


FIGURE 63.—*Index to major tributary basins.*

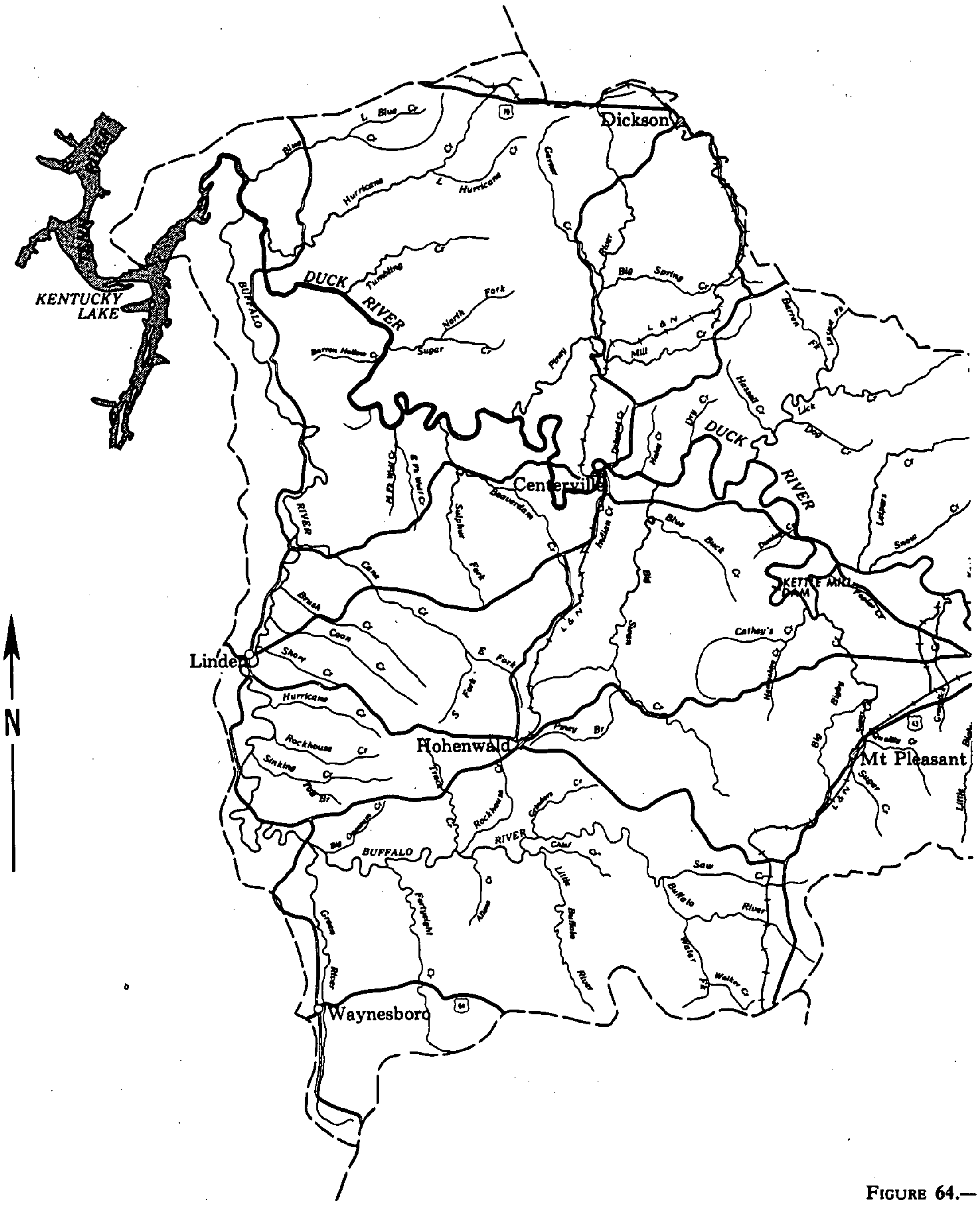
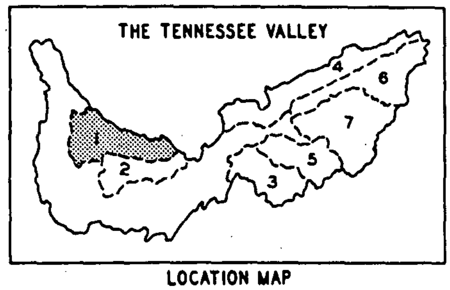

KENTUCKY LAKE
Dickson
DUCK RIVER
BUFFALO RIVER
Centerville
Linden
Hohenwald
Mt Pleasant
Waynesboro
KETTLE MILL DAM
N

FIGURE 64.—


THE TENNESSEE VALLEY
1
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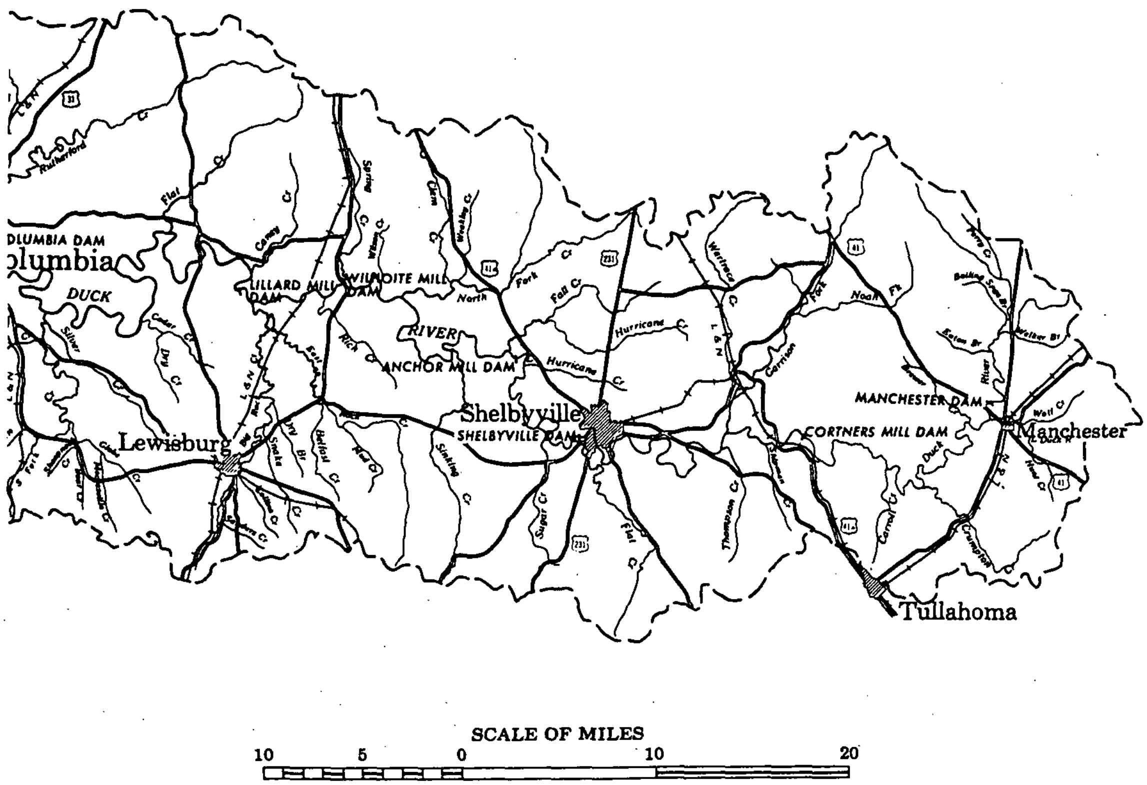
LOCATION MAP


OLUMBIA DAM
olumbia
DUCK
RIVER
LILLARD MILL DAM
WILHOITE MILL DAM
ANCHOR MILL DAM
Lewisburg
Shelbyville
SHELBYVILLE DAM
MANCHESTER DAM
CORTNERS MILL DAM
Manchester
Tullahoma
North
Fork
Fall Cr
Hurricane Cr
Noah Fk
Garrison
Thompson Cr
Sugar Cr
Flat Cr
Sinking Cr
Belfast Cr
Rich Cr
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Rutherford
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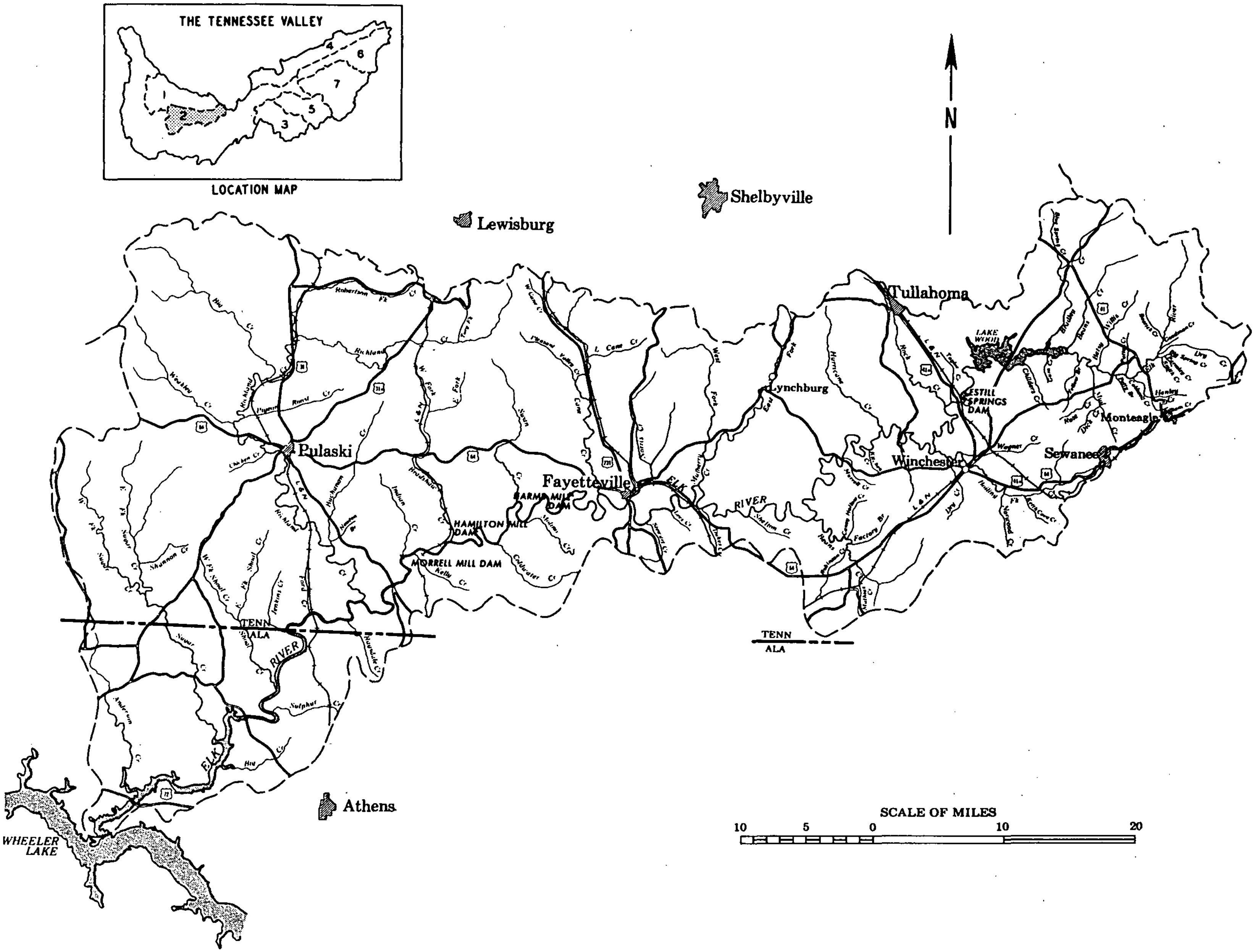


FIGURE 65.—*Elk River Basin.*

The history of flood occurrences on the Duck River extends back more than 100 years. Information on flood heights has been developed for as long as 84 years at Columbia, 70 years at Shelbyville, and 66 years at Centerville. During these years, 97 percent of the known floods above bankfull stage have occurred in the months December through April and some 75 to 80 percent in the months January through March. Even on the smaller streams of the basin the winter floods predominate.

The greatest floods on the Duck River during the period of historical record were those of March 1902, March 1929, and February 1948. None of these was a maximum over the whole river but each set records in some part of the main stream. At Manchester, near the headwaters, the 1929 flood was the record high although the 1902 crest was only 0.2 foot lower. Next downstream at Shelbyville the two floods were reversed, with the 1902 crest being 2.4 feet above that of 1929. The 1948 crest stage was in third place. At Columbia the 1948 flood moved into the lead and exceeded all others from Columbia to the mouth. Its margin over the next highest known flood was 3.7 feet at Columbia, 4.7 feet at Centerville, and 3.6 feet at the Hurricane Mills gage.

The 1948 flood was also a record on the Buffalo River which enters the Duck River below Hurricane Mills. Other very large floods on the Buffalo River occurred in March 1897, March 1902, and March 1927.

On the smaller streams of the basin there has been little uniformity in the date of occurrence of record floods. For example, Piney River experienced its greatest known stages in 1897 and 1926, Big Rock Creek in 1939 and 1955, and Little Bigby Creek in 1955 and probably 1902.

Table 8 lists crest stages and discharges for the two largest floods at several gages in the Duck River Basin.

A careful investigation was made of the damages resulting from the February 1948 flood, the largest recent flood in the Duck River Basin. The total loss was estimated to be $1,278,000. Of this amount, approximately 83 percent occurred along Duck River and its minor tributaries, and the remaining 17 percent resulted from the flood on Buffalo River. Nearly one-third of the total loss occurred in urban areas.

High floods in the Duck River Basin have a crippling effect on highway traffic in the region. In February 1948 nearly every one of the numerous bridges crossing the Duck and Buffalo Rivers was either damaged or was blocked by deep overflow on the approach roads. Traffic was disrupted for one to six days by the overflow and for considerably longer periods where bridge damage occurred.

Elk River Basin floods

The Elk River, with a drainage area of 2,249 square miles, is the second largest Tennessee River tributary west of Chattanooga. Figure 65 is a map of the basin. The stream has its origin on the eastern Highland Rim of the Cumberland Plateau in Coffee and Grundy Counties, Tennessee, only a short distance southeast of the head of Duck River. The river flows west and south from this region, entering the Tennessee River above Wheeler Dam in Alabama, 274 miles farther up the main stream than the mouth of Duck River. Like its larger companion on the north, the middle reaches of Elk River flow through a portion of the Nashville Basin, while both the upper and lower ends lie on the Highland Rim. The watershed is 34 percent forested. The topography is

TABLE 8.—*Maximum known floods at selected stations—Duck River Basin.*

Stream and station	Drainage area, square miles	Maximum known floods: Date		Gage height, feet	Estimated discharge, cubic feet per second	Discharge per square mile, cubic feet per second
Duck River:						
Below Manchester, Tennessee	107	March	1929	23.2	—	—
		March	1902	23.0	—	—
Near Shelbyville, Tennessee	481	March	1902	40.0	87,000	181
		March	1929	37.6	70,000	145
At Columbia, Tenn	1208	February	1948	51.75	61,100	51
		March	1902	48	50,000	41
Near Centerville, Tennessee	2048	February	1948	37.58	97,700	48
		March	1902	32.9	73,000	36
Near Hurricane Mills,	2571	February	1948	30.22	122,000	47
		January	1946	26.6	81,000	32
Buffalo River:						
Near Lobelville, Tennessee	707	February	1948	23.76	100,000	141
		March	1902	21.8	75,000	106
Near Flat Woods, Tennessee	447	February	1948	32.00	90,000	201

characterized generally by steep hills and rather deeply entrenched valleys. Below Fayetteville the flood plain widens from less than 1,000 feet to an average width of one-half mile.

The Elk River watershed, which is approximately 90 miles long by 25 miles wide, lies almost parallel to the path of winter storms moving northeastward across the west end of the Tennessee Valley. Richland Creek, the only large tributary, enters the Elk River 1 mile above the Prospect gaging station and 9 miles above the Alabama state line. The 488-square-mile watershed of this creek, lying entirely on the Highland Rim, extends northwestward or in a direction perpendicular to the main-river Basin. Flood flows from Richland Creek have a considerable effect on the relationship between flood heights on the upper and lower Elk River.

Only small towns are located near the river. At Fayetteville, the largest of these, backwater from the river in Norris Creek affects a number of houses and commercial establishments and a few industries. A few homes and business places at Harms, Delrose, Wheelerton, and Prospect are in the flood plain. Floods on Richland Creek affect a narrow water front area in Pulaski. Roads, bridges, and farm property are heavily damaged by floods.

There are no flood control dams on Elk River or its tributaries. Woods Reservoir, providing a water supply for the Arnold Engineering Development Center, is operated in such a way, however, as to appreciably reduce the crests of many floods along the river from Estill Springs to the junction with Richland Creek.

The investigation in the Elk River Basin extended knowledge of flood occurrences back about 130 years. Knowledge of the heights of major floods is available from 1842. As in the Duck River Basin 75 to 80 percent of the known floods have occurred in the January-March period and over 90 percent in the December-April period.

The greatest known floods on Elk River and Richland Creek were those of 1842 and 1929. From

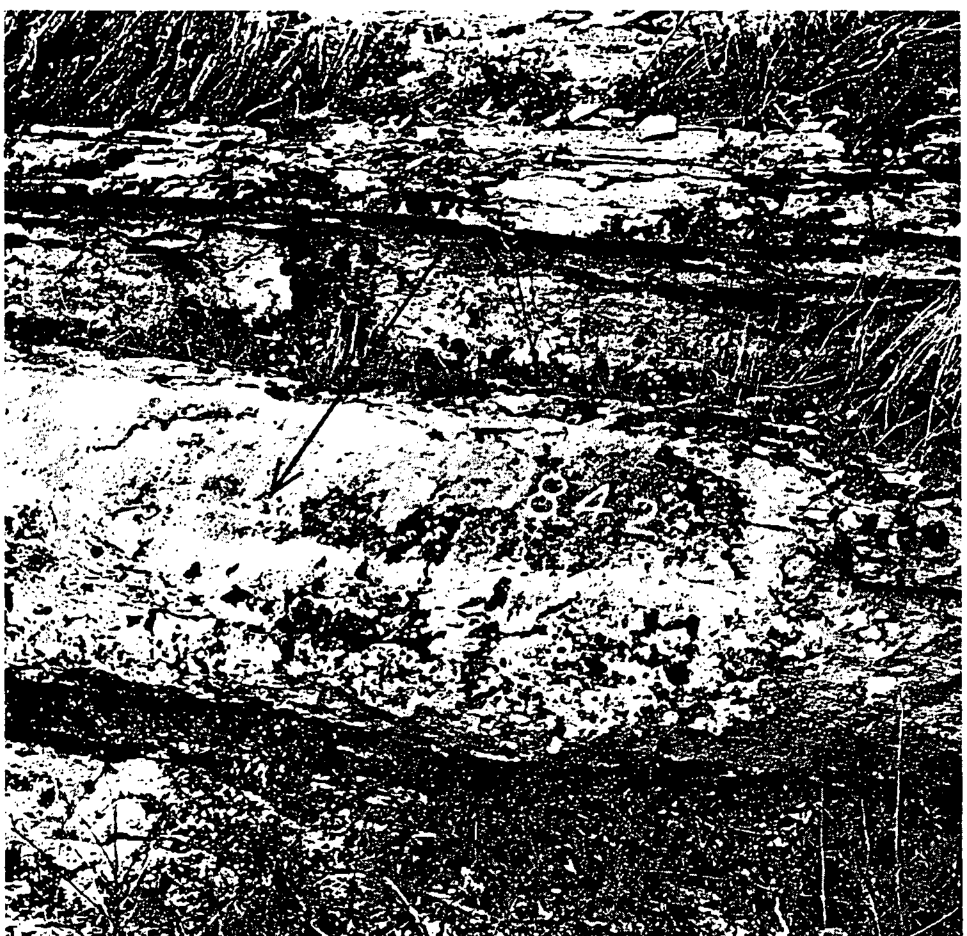


FIGURE 66.—*High water mark chiseled on bluff at Fayetteville, Tennessee—Elk River record flood of 1842.*

TABLE 9.—*Maximum known floods at selected stations—Elk River Basin.*

Stream and station	Drainage area, square miles	Maximum known floods: Date		Gage height, feet	Estimated discharge, cubic feet per second	Discharge per square mile, cubic feet per second
Elk River:						
Above Fayetteville, Tennessee	827	March	1842	27.5	46,000	56
		March	1929	27.2	44,000	54
		January	1949	27.14	36,000	43
Near Prospect, Tennessee	1784	March	1902	40.9	130,000	73
		March	1955	38.96	104,000	58
		February	1948	38.17	100,000	56
Richland Creek:						
Near Pulaski, Tennessee	366	March	1902	27.5	100,000	273
		March	1955	27.49[1]	75,000	205
		February	1948	24.58	42,600	117

1. Crest stage affected by backwater from U. S. Highway 64 bridge and embankment not present at time of 1902 flood.

Estill Springs near the headwaters of the river to the confluence with Richland Creek 47 miles below Fayetteville, the flood of 1842 exceeded all others. However, the flood of March 1929 and another great flood in January 1949 were only slightly lower over most of this reach. In Fayetteville the 1949 flood was the second highest, but upstream and downstream from the city the 1929 crest was higher than 1949. On Richland Creek at Pulaski the record flood was in March 1902, with March 1955 second and February 1948 third. This same ranking of floods prevailed on the lower 42 miles of the Elk River below the mouth of Richland Creek.

Knowledge of the record flood of 1842 is largely limited to the existence of a high water mark at Fayetteville. This mark, shown in figure 66, is a notch chiseled on a rock bluff overhanging Elk River just below the mouth of Norris Creek. The words "John N—water—1842" are cut in the rock above the mark.

Table 9 lists crest stages and discharges for the three largest known floods at several stations in the Elk River Basin.

Damage resulting from the flood of February 1948, which ranked fifth in magnitude at Fayetteville and third on Richland Creek and the lower river, was estimated at $172,000. Approximately 29 percent of this loss occurred in Fayetteville and Pulaski. The balance of the damage was largely in the rural property, crop, and highways classifications, and it is in the rural areas that floods have their greatest effect in this predominantly agricultural watershed.

Hiwassee River Basin floods

The Hiwassee River is the first major tributary of the Tennessee River above Chattanooga and one of the three major tributaries heading in the high rainfall region along the Blue Ridge. From its headwaters in northeastern Georgia, the stream flows northwestward into North Carolina and Tennessee and enters Chickamauga Reservoir 499.4 miles above the mouth of Tennessee River.

The watershed, shown in figure 67, covers 2700 square miles. It is roughly rectangular in shape, about 80 miles long by 35 miles wide. Two large tributaries, Nottely and Ocoee Rivers, draining 287 and 639 square miles, respectively, both have their origin on the slopes of the Blue Ridge. A third tributary, Valley River, with a drainage area of 117 square miles, heads on the divide between the Hiwassee and Little Tennessee watersheds.

Except at its lower end, where the river enters the relatively flat Great Valley region, the Hiwassee watershed is mountainous and rather heavily timbered. Approximately 70 percent of the basin is included in the Nantahala and Chattahoochee National Forests. Because of its location, the basin is subjected to the effects of general winter storms, which are more likely to have an accumulative effect on the lower river, and to intense summer and fall storms which are more important in the headwaters. The various tributaries have a considerable effect on the relative magnitude of floods in different portions of the river.

Several small towns are located along the Hiwassee River and its tributaries. The more important of these are Hayesville, North Carolina, on the upper Hiwassee; Murphy, North Carolina, at the junction of Hiwassee and Valley Rivers; Copperhill, Tennessee, on the Ocoee River; and Charleston, Tennessee, on the lower Hiwassee River, now a part of Chickamauga Reservoir.

The steep river slopes and high rainfall of the Hiwassee River watershed make the stream important both for power and for flood control. Before 1933, five small power dams had been built on the Hiwassee above Murphy, on the Nottely near its mouth and on the Ocoee in Tennessee and in Georgia. Between 1936 and 1943 TVA built Apalachia, Hiwassee, and

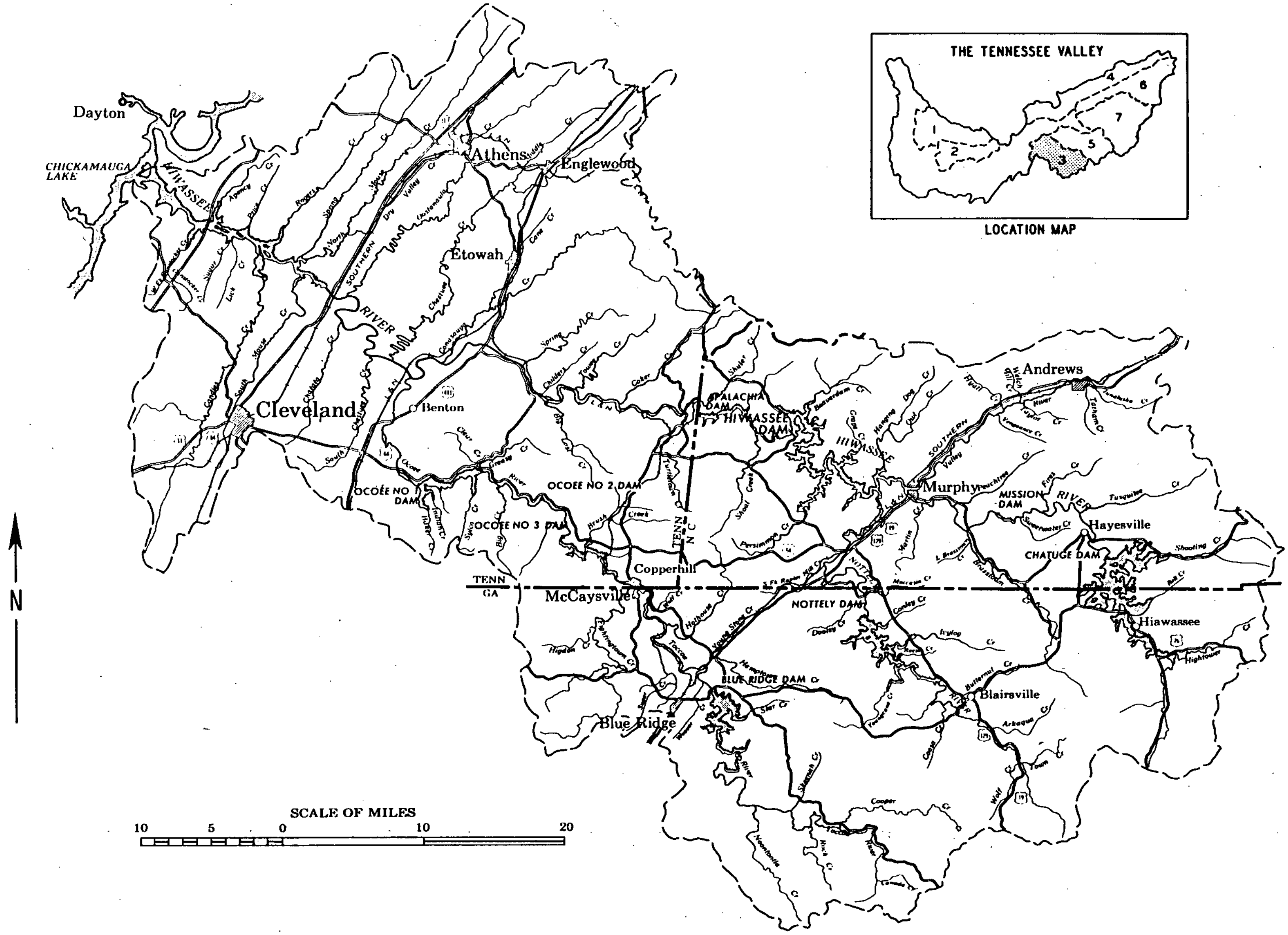


FIGURE 67.—*Hiwassee River Basin.*

TABLE 10.—*Maximum known floods at selected stations—Hiwassee River Basin.*

Stream and station	Drainage area, square miles	Maximum known floods: Date	Gage height, feet	Estimated discharge, cubic feet per second	Discharge per square mile, cubic feet per second
Hiwassee River:					
At Murphy, North Carolina	421	September 1898	17.0	29,000	69
		March 1899	17.0	29,000	69
		March 1886	16.5	28,000	69
At Reliance, Tennessee	1223	September 1898	31	90,000	74
		March 1886	29	78,000	64
		November 1906	28	72,000	59
Nottely River:					
Near Blairsville Georgia	74.8	September 1898	14.5	—	—
		July 1916	13.5	—	—
		July 1938	12.0	—	—
Valley River:					
At Tomotla, North Carolina	104	September 1898	22.4	32,000	308
		November 1906	22.2	31,000	298
Ocoee River:					
At Parksville, Tennessee	595	November 1906	27	65,000	109
		April 1920	24	42,500	71
At Copperhill, Tennessee	352	November 1906	17.7	—	—
		September 1898	16	—	—

Chatuge Dams on Hiwassee River, Nottely Dam on Nottely River, and Ocoee No. 3 Dam on Ocoee River. At the present time the Hiwassee River is one of the most completely controlled streams in the Tennessee Valley.

Major floods on the Hiwassee River and its tributaries occur most frequently in the months December through April, but some of the greatest known floods have been experienced in the summer and fall months. On Hiwassee River the largest flood at and above Hayesville, North Carolina, was in October 1898, when the river was reported to have been "five feet higher than ever known before." A few miles downstream at Murphy, the maximum known flood was experienced on September 3, 1898. Fed by record floods also on Valley and Nottely Rivers, the September 1898 flood was a maximum all the way downstream to Charleston, Tennessee. A flood in March 1899 was nearly as high at Murphy and one in March 1886 ranked third. Farther down the Hiwassee River the 1886 flood moved up to second place. Above the present Chatuge Dam near Hayesville, it appears likely that the flood of June 16, 1949, approached very close to the record of October 1898.

On Valley River the rise of September 1898 was highest, and the flood of November 1906 was only slightly lower. Nottely River experienced its greatest known flood down to the site of present Nottely Dam in September 1898. *The Cherokee Scout* of Murphy, North Carolina, reported in its issue of September 13, 1898, that "Notla River (on September 3) was the highest known in 70 years, and Valley and Hiwassee Rivers were the highest known since 1840." High water marks indicate that major floods also occurred on the upper Nottely River in July 1916 and July 1938. Along the lower Nottely River the flood of November 1906 was the highest.

The storms in the fall of 1898 apparently were less severe in the southwestern part of the basin and the maximum flood on the Ocoee River occurred in November 1906. The September 1898 flood ranked second at and above Copperhill. High water marks show that a major flood occurred in April 1920 at and below Parksville.

Table 10 lists crest stages and discharges for the two or three of the highest known floods at several locations in the watershed.

The effects of floods in the Hiwassee River Basin are now confined largely to the headwater reaches above Chatuge, Nottely, and Blue Ridge Dams and possibly to the Copperhill, Tennessee, vicinity on Ocoee River. Flood losses are largely to rural property, crops, and highways. The high velocities of these mountain streams cause severe land scour when flooding occurs. In general, flood losses in the Hiwassee River Basin have been small compared to those suffered in the western part of the Tennessee Valley or in the more highly developed French Broad River watershed to the east.

Clinch and Emory River Basin floods

The Clinch River watershed, shown in figure 68, is the longest and narrowest of any of the major tributaries of the Tennessee River. From Kingston,

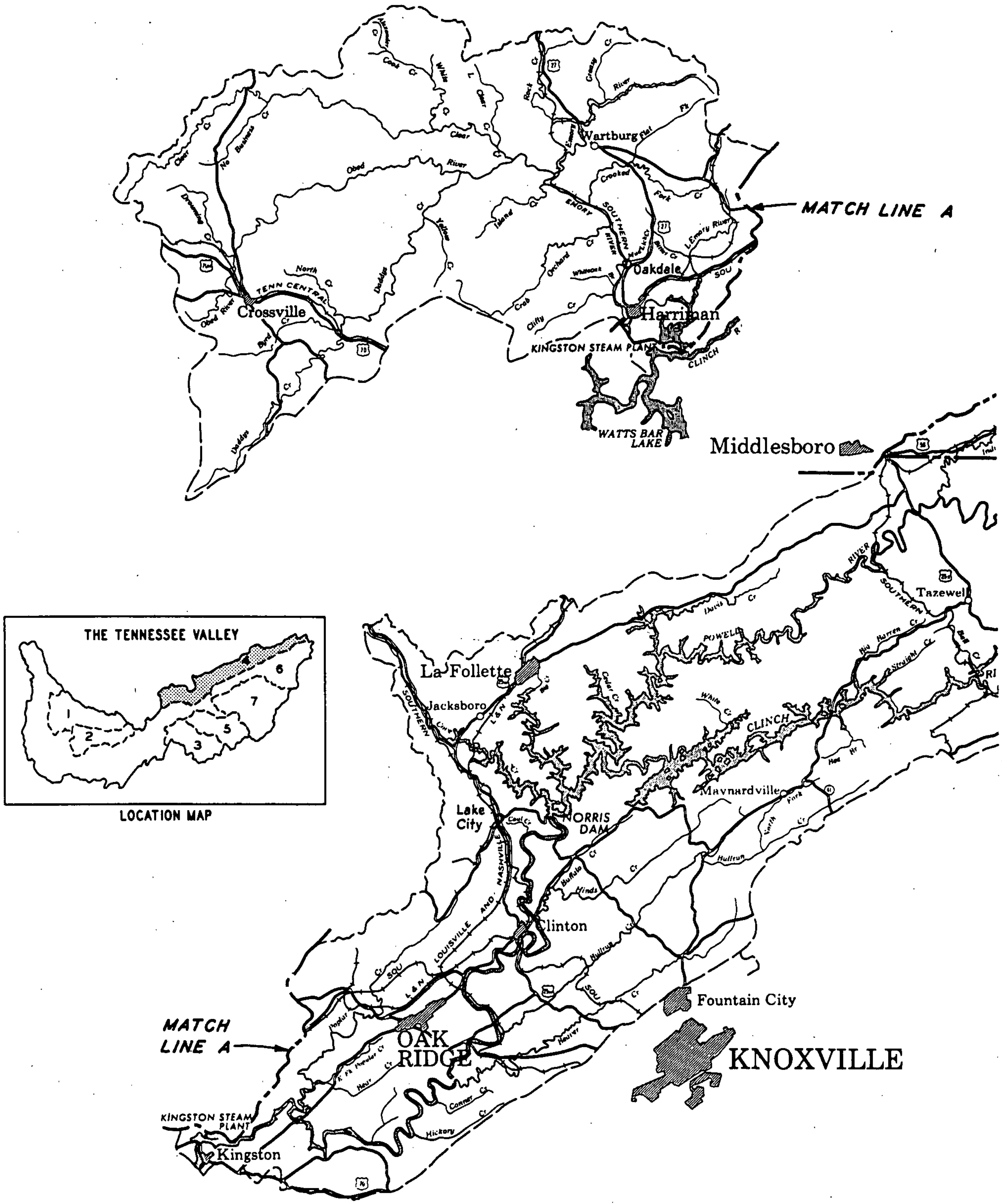
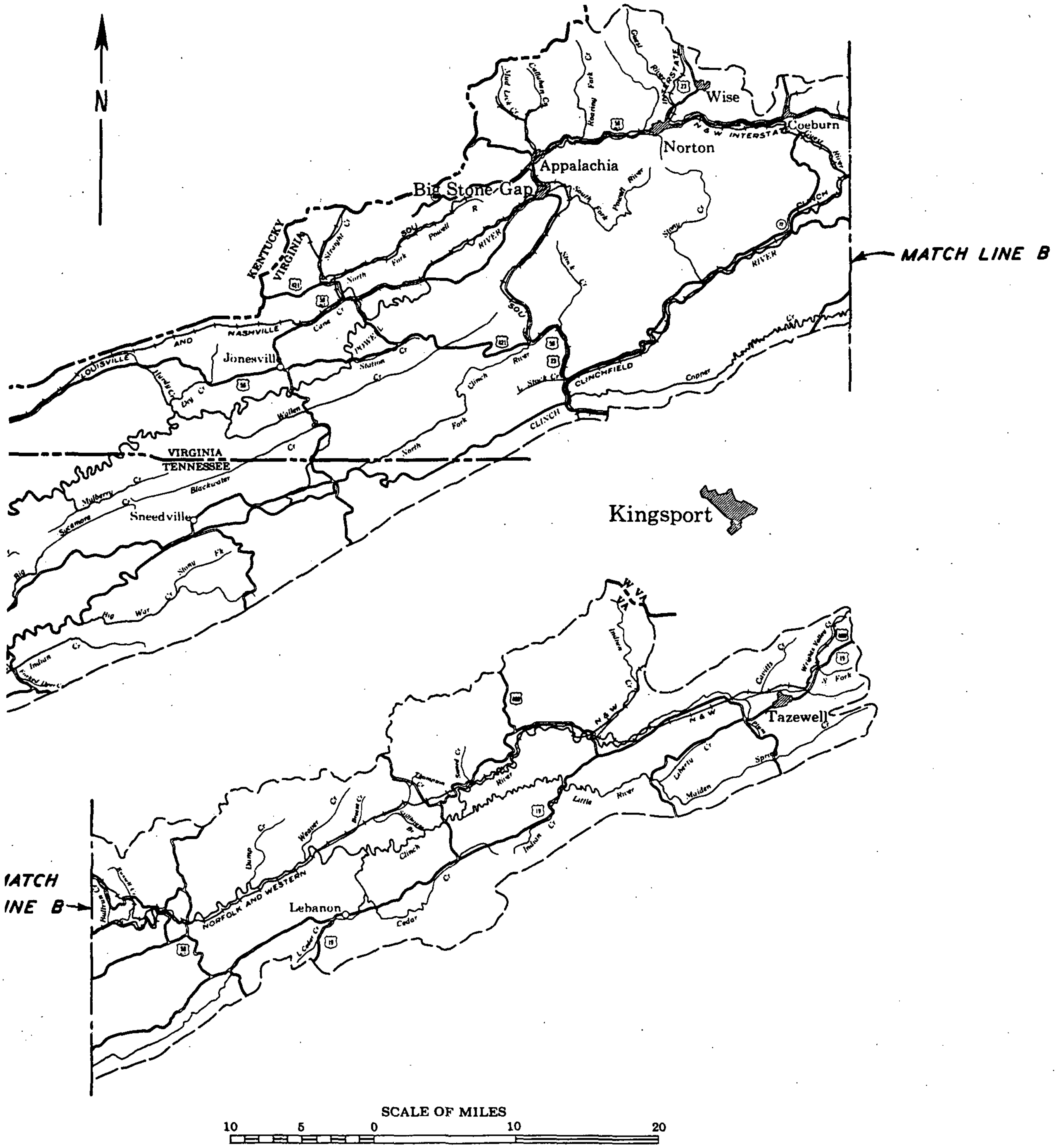


FIGURE 68.—*Cli*


N
Wise
Coeburn
Norton
Appalachia
Big Stone Gap
KENTUCKY
VIRGINIA
MATCH LINE B
Jonesville
VIRGINIA
TENNESSEE
Sneedville
Kingsport
CLINCHFIELD
NORFOLK AND WESTERN
Tazewell
Lebanon
MATCH LINE B
SCALE OF MILES
10 5 0 10 20

r Basin.

Tenneseee, the Clinch Basin follows the Great Valley northeastward along the base of the Cumberland Plateau escarpment for nearly 200 miles through Tennessee and Virginia. The 4,413-square-mile drainage area averages less than 25 miles wide and, exclusive of the Emory River, its width never exceeds 30 miles. The topography is characterized by parallel mountain ridges that extend northeast and southwest and confine the streams in relatively straight courses between them. The ridge slopes are steep and bottom lands are generally narrow. Approximately 48 percent of the basin above Norris Dam is timbered and a considerable portion of the headwater area is national forest. The topography of the lower river basin differs from that upstream in that the Valley broadens considerably and has wider flood plains bordered by rolling hills.

The Clinch River has only two important tributaries. Powell River, with a drainage area of 938 square miles, enters the Clinch River 9 miles above Norris Dam. Its watershed, lying north of and parallel to that of the Clinch River, is quite similar in character to the main stream area. Emory River, joining the Clinch River only 4.4 miles above its mouth, has an 865-square-mile drainage area which is quite different from that of the parent stream. The Emory River Basin, shown in figure 68, is located almost entirely on the Cumberland Plateau, and its tributary streams, in a fan-shaped pattern, flow in deep, narrow valleys cut in the surface of the plateau. Only the lower reach of the river, below Harriman, Tennessee, is in the Great Valley province.

The high mountains of the Blue Ridge region shield the Clinch Basin from the tropical hurricane storms that cause devastating summer and fall floods on the Hiwassee, Little Tennessee, and French Broad Rivers. Nearly 90 percent of the floods on the Clinch River and 75 percent of the floods on the Emory River have occurred during December through April, and only minor rises have been experienced during the summer and fall. The long, narrow shape of the Clinch River watershed is reflected noticeably in the flood experience. The more or less isolated position of the Emory River Basin with respect to the rest of the watershed causes its flood record to vary considerably from that of the main stream.

Numerous mill dam installations are found on the small tributaries, and a few are located on the main rivers. The only major regulatory structure is Norris Dam, completed by TVA in 1936.

The only towns located in the proximity of the main streams are Big Stone Gap, Virginia, at the junction of the Powell and South Fork Powell Rivers; Cleveland and St. Paul, Virginia, and Clinton, Tennessee, on the Clinch River; and Harriman, Oakdale, and Gobey, Tennessee, on the Emory River. Clinton is located below Norris Dam.

The greatest known flood on the Clinch River above the confluence with the Powell River occurred in February 1862. Old residents above Sneedville were of the opinion that this was the highest flood that has occurred since the earliest date of settlement along the river. In the vicinity of Clinton, an authentic mark was found indicating that the maximum flood on the lower river occurred in March 1826.

The second greatest flood on the upper Clinch River and the maximum known flood on Powell River occurred in January 1918. This flood, commonly called the "ice tide" by people living along the river, occurred during the extremely severe winter of 1917-1918 when snow accumulated to depths of 20 to 25 inches in the upper part of the basin. A combination of heavy rain, rapidly rising temperature, and frozen ground resulted in the high runoff that caused the flood. Late in January 1957 large floods occurred on the upper Clinch and Powell Rivers. At the Clinch River station above Tazewell, Tennessee, the crest of this flood was only 0.3 foot below that of 1918.

At Clinton and below, the floods of February 1862 and March 1886 share second rank after the March 1826 flood, while the flood of January 1918 is in third place. The flood situation on the lower river has been greatly altered by the operation of Norris Reservoir which has a flood storage reservation of about 1.5 million acre-feet during the January-March period. However, large and damaging floods are still possible from the uncontrolled drainage basin below Norris Dam.

On Powell River, a great flood in January 1946 ranked second only to the 1918 maximum. At the gaging station near Arthur, Tennessee, which is just above Norris Reservoir backwater, the 1946 flood was only a few hundredths of a foot below the estimated crest in 1918. At Big Stone Gap, information obtained from newspaper accounts indicated that a flood on the Powell River in 1840 was approximately the same height as the record stage of 1918 at that location.

The Emory River experienced its greatest flood on March 23, 1929, when an unprecedented storm occurred on the Cumberland Plateau. Rainfall totalling 6 to 11 inches fell in 12 to 15 hours on the Emory River watershed. The resulting flood, which was far above any others known to have occurred on the river, took a toll of 20 lives and caused approximately $3,500,000 damage. Information obtained on the Obed River, a western tributary of Emory River, showed that the 1929 flood on that stream was the highest in at least 100 years. The second ranking flood in most of the Emory River Basin occurred in March 1902. The crest of this flood, which was the oldest for which definite data were found, was 11 to 12 feet under the 1929 crest at Oakdale and Harriman and 6 to 8 feet lower than the 1929 crest at the upstream stations. At Wartburg, Tennessee, on the upper Emory River, the 1902 rise was exceeded slightly by a flood in February 1939. The flood of February 1948 was third highest on the Emory River from the mouth of Obed River to Oakdale but

TABLE 11.—*Maximum known floods at selected stations—Clinch River Basin.*

Stream and station	Drainage area, square miles	Maximum known floods: Date		Gage height, feet	Estimated discharge, cubic feet per second	Discharge per square mile, cubic feet per second
Clinch River:						
Above Tazewell, Tennessee	1474	February	1862	24	66,000	45
		January	1918	21.3	54,000	37
		January	1957	21.0	51,100	35
At Clinton, Tennessee	3056	March	1826	43.5	130,000	43
		February	1862	41.3	117,000	38
		March	1886	41.3	117,000	38
Powell River:						
At Big Stone Gap, Virginia	112	January	1918	15.7	33,000	295
		January	1946	9.8	16,500	147
		January	1957	9.67	11,000	98
Near Arthur, Tennessee	685	January	1918	27.2	33,000	48
		January	1946	27.15	33,000	48
Emory River:						
Near Waterburg, Tennessee	83.2	March	1929	32	30,000	361
		February	1939	25.62	18,000	225
At Oakdale, Tennessee	764	March	1929	42.3	195,000	255
		March	1902	31.8	110,000	144

ranked below the 1939 and 1902 crests on the upper Emory River.

Table 11 lists crest stages and discharges for the two or three highest known floods at several locations in the Clinch and Emory Rivers watersheds.

Damage from floods on the Clinch and Powell Rivers is confined chiefly to crops and property on bottom land farms. Highways and railroads suffer minor damages, but most settlements are well above flood level and urban losses are small. Damage from floods in the Emory Basin occurs largely to industries and other urban losses in Harriman and Oakdale, and to highways and railroads. The floods of February 1939 and February 1948 caused damage ranging between $150,000 and $200,000 over the Emory River Basin.

Little Tennessee River Basin floods

The Little Tennessee River is the third in order of size and the farthest upstream of the three major tributaries entering the Tennessee River between Chattanooga and Knoxville. Its watershed, covering 2,627 square miles, lies to the north and east of the Hiwassee River watershed and is very similar in character to that basin. Figure 69 is a map of the Little Tennessee Basin.

Rising on the slopes of the Blue Ridge in North Carolina and Georgia, the Little Tennessee River and its tributaries follow a tortuous path through some of the highest mountains in the eastern United States. Only the lower 40 miles of Little Tennessee River below Calderwood Dam are in the relatively flat Great Valley province. Eighty percent of the Basin is still covered by forest, and nearly all the North Carolina portion of the Great Smoky Mountains National Park is in the watershed.

Of the five principal tributaries, the Tellico River with a drainage area of 285 square miles is almost entirely in Tennessee. This stream enters the main river 19 miles above its mouth. Cheoah River, with a 215-square-mile basin, flows into the Little Tennessee just below Cheoah Dam and just above the North Carolina-Tennessee state line. The Tuckasegee, Nantahala, and Cullasaja Rivers, with drainage areas of 734 square miles, 175 square miles, and 93 square miles, respectively, all rise on the slopes of the Blue Ridge and parallel the direction of the main stream. Cullasaja River enters the Little Tennessee near Franklin, North Carolina, while the other two are tributary to Fontana Reservoir.

Like the Hiwassee River, the Little Tennessee River headwaters are exposed to hurricane storms of tropical origin, to intense, more localized summer and fall convection type storms, and to the general winter storms that sweep across the Tennessee Valley. The varying exposure of the tributaries makes for wide differences in rainfall distribution so that no one storm has produced a maximum flood over the whole watershed. Floods occur most frequently in the December-April period but some of the great floods of record have occurred in the summer and fall months.

Several small towns are situated on the streams. Those affected to some extent by floods are Franklin, North Carolina, on the Little Tennessee River; Dillsboro, Whittier, Cullowhee, and Bryson City, North Carolina, on the Tuckasegee River; and Tellico Plains, Tennessee, on the Tellico River.

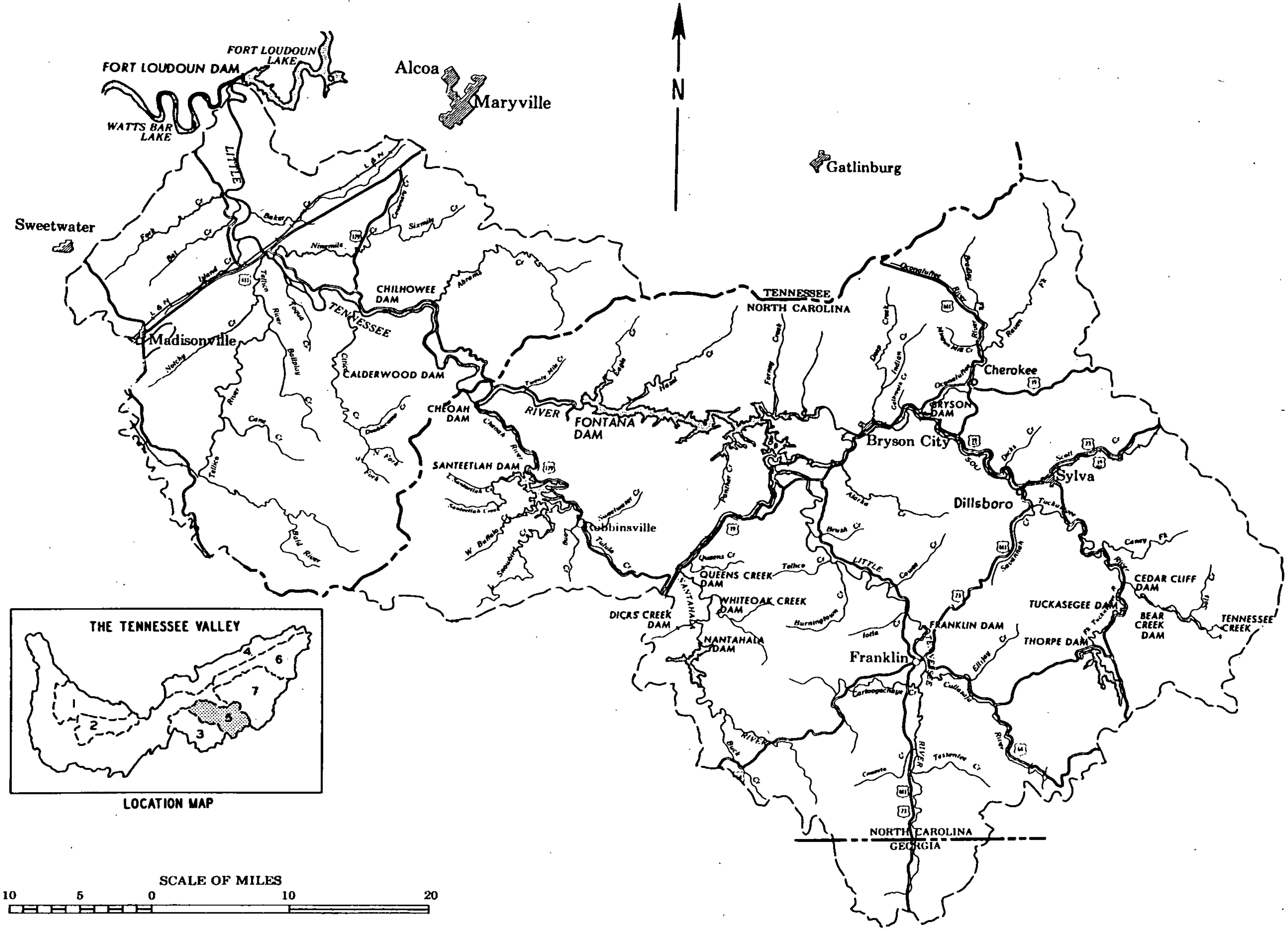


FIGURE 69.—*Little Tennessee River Basin.*

The Aluminum Company of America began development of the power resources of the Little Tennessee River 20 years or more before the TVA Act was passed by Congress. The principal dams built by the company were Calderwood and Cheoah Dams on the Little Tennessee River and Santeetlah Dam on the Cheoah River. Other early power dams in the area were built on the Little Tennessee River below Franklin and at the mouth of the Oconaluftee River near Bryson City. These are now operated by the Nantahala Power and Light Company, an Aluminum Company subsidiary. Since 1940 the Aluminum Company has completed two storage dams, Thorpe on the West Fork Tuckasegee River and Nantahala on the Nantahala River, and five small projects in the Tuckasegee and Nantahala River Basins. In 1957, Chilhowee Dam, located on the Little Tennessee River 10 miles downstream from Calderwood Dam, was placed in operation by the Aluminum Company. In 1944 TVA completed Fontana Dam, a large flood control and power project on the Little Tennessee River above Cheoah Dam backwater.

The great flood of October 1898, which was a maximum on the upper Hiwassee River, was also the largest known on the Little Tennessee River headwaters. From above the Cullasaja River down about to the junction with Nantahala River this flood exceeded all others in the memory of old residents. There is some question as to the date of the second ranking flood in this upper reach. In the Franklin, North Carolina, *Press* for October 12, 1898, there appears a statement attributed to Mr. Sam Hall to the effect that a flood in February 1875 was only 10 inches below the October 1898 crest. In the same paper, however, Mr. Tom Downs is quoted as saying that the river in October 1898 "was 3 feet and 2 inches higher at his place than ever before known since his recollection." Two floods since 1898 have exceeded any others in the upper reach. These were the floods of August 30, 1940, and June 16, 1949. The latter flood was the greater above the mouth of the Cullasaja River, but the 1940 flood exceeded the recent rise at and below Franklin.

In the middle reaches of Little Tennessee River, at the site of Fontana Dam, the pattern of high floods changes considerably. Here the great Valley-wide flood of March 1867 was a maximum and the flood of May 1840, coming largely from the Tuckasegee River, was the second highest. Another spring flood, in March 1886, ranked third in this reach.

Near the mouth of Little Tennessee River the flood of March 1867 was still a maximum, and the floods of March 1886 and February 1875 were in second and third place, respectively.

On the upper Tuckasegee River down to Dillsboro, North Carolina, the flood resulting from the storm of August 29-30, 1940, was the highest ever known. Rainfall during this storm totalled 6 to 13 inches in 24 hours over the East, West, and Caney Forks of Tuckasegee River, and devastating floods resulted on these headwater streams. A flood 100 years earlier, in May 1840, was second highest in this upstream reach. From Dillsboro downstream these two floods exchanged places, and at Bryson City, North Carolina, the 1840 crest was some 4 feet higher than the late-August flood of 1940. It is probable that a very high flood also occurred in 1840 on Oconaluftee River, a Tuckasegee River tributary heading in the Great Smoky Mountains National Park and entering the main river above Bryson City. The greatest known floods on Oconaluftee River occurred in March 1867, November 1906, and March 1913. All reached approximately the same height in the Cherokee, North Carolina, vicinity.

Maximum flood dates on the smaller tributaries of Little Tennessee River varied with each stream. The August 30, 1940, rise was the greatest on Cullasaja River; the flood of March 1917 was the largest on the lower Nantahala River since its settlement 80 or more years ago; the flood of February 1875 exceeded any others known to old settlers on the Tellico River, and the flood of November 1906 was a maximum on the Cheoah River. On the upper Nantahala River, in the vicinity of Rainbow Springs, North Carolina, people 60 to 70 years old said that the flood of June 16, 1949, was the greatest they had ever seen.

Table 12 lists crest stages and discharges for some of the highest known floods at selected locations in the Little Tennessee River watershed.

The effects of floods are more severe along the well-developed Tuckasegee River than elsewhere in the Little Tennessee River Basin. There is little or no bottom land on the Little Tennessee River itself except in the reach at and above Franklin and in the Great Valley portion below Calderwood Dam. Much of the Nantahala River is in a deep gorge, and there is little along it that can be damaged except highway structures. Conditions along Cheoah and Cullasaja Rivers are similar. The upper Tellico River flows through an extremely rugged national forest area with practically no bottom land. Below Tellico Plains, however, this river enters the Great Valley region, and some farm land is subjected to flooding.

By comparison with the other streams in the watershed, the Tuckasegee River is well settled, with a number of small towns and villages scattered along its banks. Small industries, businesses, homes, highways, railroads, utilities, and agricultural interests all suffer from large floods on this stream. In the flood of August 30, 1940, the total loss in the Tuckasegee River Basin exceeded $400,000 as compared to approximately $30,000 on the upper Little Tennessee River in the same flood. The flood of June 1949, which was near a record on the Little Tennessee River above Franklin, caused damage on that river and its small headwater tributaries totalling a little more than $100,000.

TABLE 12.—*Maximum known floods at selected stations—Little Tennessee River Basin.*

Stream and station	Drainage area, square miles	Maximum known floods: Date		Gage height, feet	Estimated discharge, cubic feet per second	Discharge per square mile, cubic feet per second
Little Tennessee River:						
At Needmore, North Carolina	436	October	1898	13	28,000	64
		August	1940	11.5	22,000	50
		June	1949	11.10	20,200	46
At McGhee, Tennessee	2443	March	1867	39.5	165,000	68
		March	1886	38.8	160,000	65
Tuckasegee River:						
At Bryson City, North Carolina	655	May	1840	20	90,000	137
		March	1867)	17	65,000	99
		June	1876)			
		August	1940	15.96	61,600	94
Nantahala River:						
Near Rainbow Springs, North Carolina	51.9	June 16,	1949	9.70	6,300	121
Tellico River:						
At Tellico Plains, Tennessee	118	February	1875	14.5	19,800	168
		March	1867	14	18,200	154
		January	1957	13.60	17,500	148
Oconaluftee River:						
At Cherokee, North Carolina	131	March	1867	13	—	—
		November	1906	13	—	—
		March	1913	13	—	—

Holston River Basin floods

The Holston River joins with the French Broad River just above Knoxville to form the Tennessee River. Its drainage basin is shown in figure 70. The river is in some respects a companion stream to the Clinch River, which it parallels. Practically the entire drainage basin, except for the Watauga River, lies in the Great Valley region. The watershed, again exclusive of Watauga River, is long and very narrow, extending 165 miles from Knoxville northeastward into Virginia, and never exceeding 30 miles in width. Two major tributaries, the North Fork and South Fork, lie in a region of long, steep-sided, parallel ridges which hold the streams in comparatively straight northeast-southwest valleys.

The Watauga River, an important tributary of the South Fork Holston River, has a fan-shaped drainage basin which extends southeastward into the Appalachian Mountain area. Its headwater region, on the slopes of the North Carolina Blue Ridge, is similar in character to those of the upper Little Tennessee, Hiwassee, and French Broad Rivers. Only 46 percent of the Holston River watershed is forested, and much of this forest is in the Watauga Basin.

The drainage area of the Holston River at its mouth is 3,776 square miles. The South Fork drains an area of 2,048 square miles while the North Fork, which joins it below Kingsport to form the main Holston, has a watershed of 729 square miles. The Watauga River contributes the flow from 869 square miles to the South Fork. Approximately 75 percent of the entire Holston River watershed is above the junction of the two forks.

Like the Clinch River watershed, the Holston Basin is largely protected by the mountains from the effects of summer and fall hurricane storms, and its major floods have occurred in the winter and spring months. The position of the Watauga River headwaters, however, exposes this stream to the full effects of such storms.

During Colonial times the Holston River area was one of the earliest in the Tennessee Valley to be settled extensively. Because the stream penetrates a region of substantial mineral resources, a number of mining and processing plants have been located near the streams. Several large towns have developed as a result of industrial expansion. Two of these, Elizabethton on the Watauga River and Kingsport on the South Fork Holston River, are subject to considerable flood damage from these streams and also from tributaries entering the main stems. Other sizeable towns affected by floods are Saltville, Virginia, on the North Fork Holston River; Marion, Virginia, on the Middle Fork Holston River; Bristol, Tennessee-Virginia, on Beaver Creek; Roan Mountain, Tennessee, on Doe River; Damascus, Virginia, on Laurel Creek; and Morristown, Tennessee, on Turkey Creek.

Prior to the entrance of TVA into the Valley, the only power development in the Holston watershed, other than small mill dams, was Wilbur Dam of the East Tennessee Light and Power Company. This dam, with 327 acre-feet of storage behind it, is on the Watauga River about 5 miles east of Elizabethton. Since 1940, TVA has built five dams in the watershed. Four are for flood control and power production: Cherokee Dam on the lower Holston River, completed in 1941; Watauga Dam on the Watauga River above Wilbur Dam, completed in 1948; South Holston Dam on the South Fork Holston River near Bristol, Tennessee, completed in 1949; and Boone Dam on the South Fork Holston River just below the confluence with the Watauga River, completed in 1953. These four dams provide substantial control of floods at Elizabethton and Kingsport, and on the lower river. The fifth, Fort Patrick Henry Dam located just downstream from Boone Dam, was completed in 1953. This project has no flood control function.

Seventy to 75 percent of the overbank floods on the larger streams in the basin occurs in the December through April period. Outstanding floods also occur in the summer and fall months, particularly on the Watauga and South Holston Rivers.

The maximum known flood on the Holston River below Kingsport occurred in March 1867. Old residents interviewed during the flood history investigation asserted that at the time of the 1867 flood no one then living could recall having experienced a higher one. Mr. J. P. Burem, who lives near the head of Cherokee Reservoir and whose ancestors have kept a family diary since they settled along the Holston in 1776, said that at the time of the 1867 flood the entry read, "Water everywhere. Greatest flood every known." The second and third highest floods in this 142-mile reach of the river were, interchangeably, the floods of February 1875 and May 1901. The 1901 flood, which was largely the result of a record high water on the South Fork, was the higher of the two down to the vicinity of Morristown, Tennessee. Below this point, a few scattered marks indicate that the February 1875 was slightly higher. Both were 4 feet or more below the 1867 crest over most of the reach.

On the South Fork above the mouth of Watauga River the flood of March 1867 was a maximum by a substantial margin. The flood of May 1901 was second, and the February 1875 crest was third.

On the Watauga River the floods of August 13, 1940, and May 21, 1901, shared first rank. Above the junction with the Doe River at Elizabethton, the 1940 flood exceeded all other known floods. At Butler it was more than 4 feet higher than in May 1901, the previous maximum. At and below Elizabethton, however, a great flood on the Doe River in May 1901 caused this older flood to exceed the 1940 crest. The effect of the tremendous flow from the Watauga on the already high flow in the South Fork in May 1901 caused the flood to be a maximum on the lower reaches of the South Fork of Holston

Table 13.—*Maximum known floods at selected stations—Holston River Basin.*

Stream and location	Drainage area, square miles	Maximum known floods: Date		Gage height, feet	Estimated discharge, cubic feet per second	Discharge per square mile, cubic feet per second
Holston River:						
Near Jefferson City, Tennessee	3429	March	1867	59	150,000	44
		February	1875	53	114,000	33
		May	1901	53	114,000	33
North Fork Holston River:						
Near Gate City, Virginia	672	February	1862	22.5	54,000	80
		March	1867	22	52,000	77
South Fork Holston River:						
At Kingsport, Tennessee	1931	May	1901	23	110,000	57
		March	1867	22.5	104,000	54
At Bluff City, Tennessee	813	March	1867	21	51,000	63
		May	1901	19	43,000	53
Watauga River:						
At Elizabethton, Tennessee	692	May	1901	21	76,000	110
		August	1940	20.87	75,100	109
Near Sugar Grove, North Carolina	90.8	August	1940	29.6	50,800	560
Doe River:						
At Elizabethton, Tennessee	137	May	1901	10.5	39,000	285

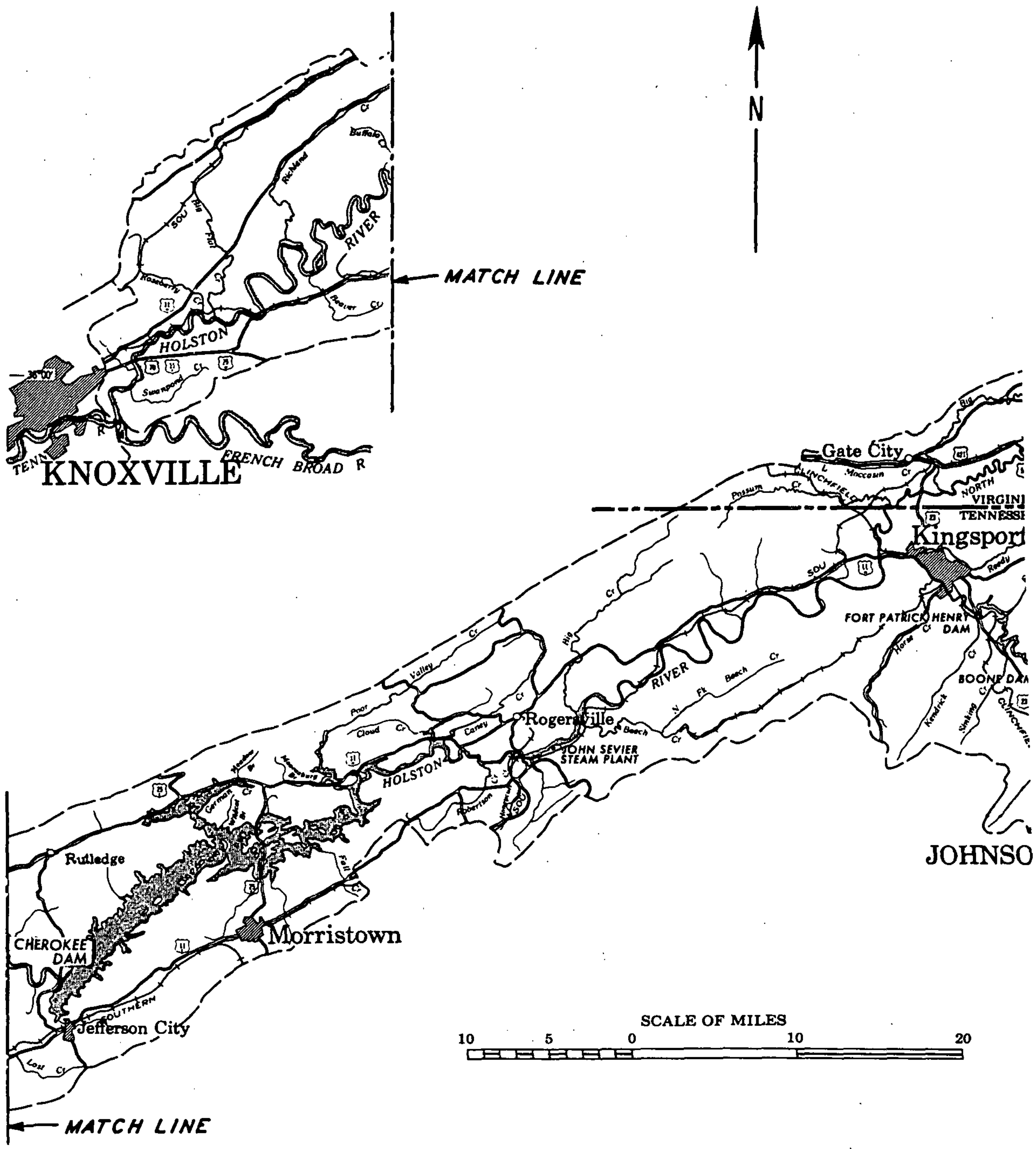
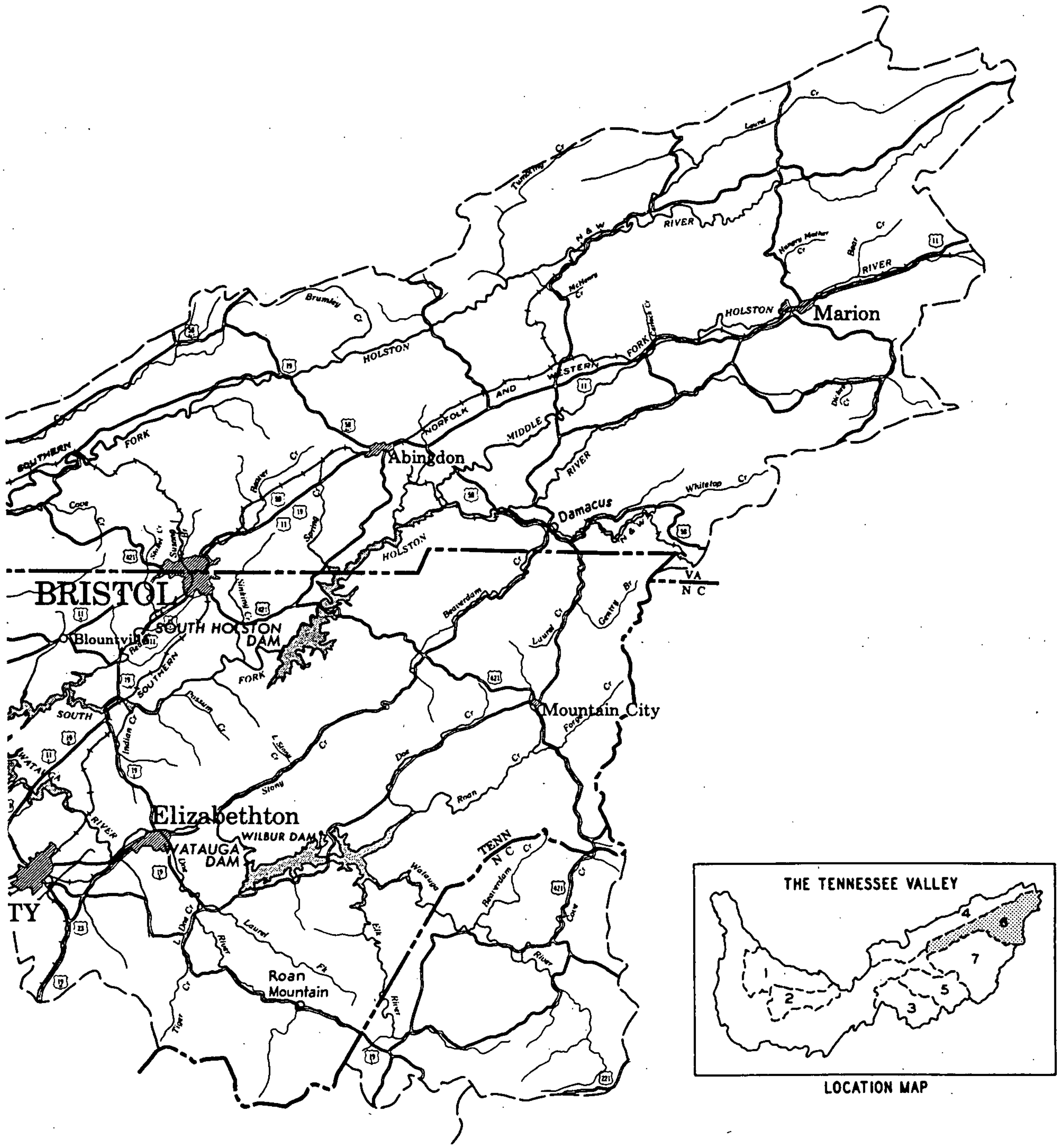


FIGURE 70.—*Hols.*


Marion
HOLSTON
RIVER
Abingdon
NORFOLK AND WESTERN
MIDDLE
FORK
Damacus
BRISTOL
SOUTH HOLSTON DAM
Blountville
Mountain City
Elizabethton
WILBUR DAM
WATAUGA DAM
Roan Mountain
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TY
THE TENNESSEE VALLEY
LOCATION MAP

er Basin.

River. The March 1867 flood was but little lower at Kingsport.

The North Fork Holston River, which is adjacent to the Clinch River watershed, was raised to its maximum level by the storm of February 1862. This flood was also a maximum on the upper Clinch River. The second greatest flood on the North Fork occurred in March 1867 and the third highest in February 1875.

Table 13 lists crest stages and discharges for several of the highest known floods at selected locations in the Holston River watershed.

Great floods of the magnitude of those in 1867, 1901, and 1940 have caused heavy losses to industries, businesses, and homes in Kingsport, Elizabethton, Saltville, and smaller towns, and to roads and farm property in the rural areas. The flood of August 13-14, 1940, in the Watauga River Basin resulted in losses totalling approximately one and one-quarter million dollars. Thirty-one percent of the damage was to highways; 16 percent to railroads; 19 percent to industry; 19 percent to urban and rural homes, crops, and property; 11 percent to utilities; and the balance to small businesses and municipalities. The flood in the remainder of the Holston River Basin caused damage totaling about $100,000, half of which was crop damage.

French Broad River Basin floods

The French Broad River is the largest of the Tennessee River tributaries, draining a basin of 5,124 square miles. Figure 71 is a map of the basin. The river is important not only because of its effect on floods downstream but also because of the heavy damage it causes in its upper reaches. Since 1943 the contribution of the French Broad River to Tennessee River flood flow has been controlled by Douglas Dam, but the upper French Broad River remains a problem which has become increasingly critical as the region's industry and agriculture develop.

The French Broad River watershed in Western North Carolina and a small part of the basin in Tennessee lie in the Appalachian Mountain region. This portion of the basin, constituting approximately 60 percent of the whole, is located between the Little Tennessee River area on the southwest and the Watauga River area on the northeast. The lower part of the French Broad Basin is in the Great Valley region with comparatively low relief and flat slopes. Some 57 percent of the whole basin is forested.

The river rises on the slopes of the Blue Ridge southwest of Asheville, North Carolina. Here four headwater forks plunge down from elevations of 3000 to 5000 feet in the Pisgah National Forest to form the main stream at Rosman, North Carolina. Below Rosman the river enters a relatively broad, flat plain through which it flows northward to Asheville. In this reach, Davidson River, Mills River, and Hominy Creek, with sources on the mile-high Pisgah Ridge, enter on the west while Little River, Mud Creek, Cane Creek, and Swannanoa River, heading on the lower levels of the Blue Ridge, enter on the east. These streams, with drainage areas ranging from 47 square miles to 133 square miles, together with the main river, drain the area known as the Upper French Broad Region.

From Asheville to Marshall, North Carolina, the river flows in a narrow valley that is 200 to 400 feet deep. Flood plains are narrow, and the river flows on bedrock. Ivy River, with a drainage area of 161 square miles, is the largest tributary in this reach. Near Marshall the stream enters a precipitous gorge, 400 to 1,000 feet deep, from which it emerges about 2 miles upstream from Bridgeport, Tennessee. Several tributaries, flowing in equally precipitous valleys, enter the river in the reach. The largest is Laurel Creek with a watershed area of 132 square miles.

Near Bridgeport, the French Broad River enters the Great Valley of East Tennessee and flows generally westward through Douglas Reservoir to its confluence with the Holston River above Knoxville. It is in this region of gentle gradients and extensive flood plains that the three largest tributaries join the French Broad. The Nolichucky River, with a drainage area of 1,756 square miles, and the Pigeon River, with an area of 689 square miles, both enter above Douglas Dam. Their watersheds lie east and west. respectively, of the main stream area and are quite similar in character. The third tributary, Little Pigeon River, enters the parent stream a few miles below Douglas Dam. Its source is on the highest slopes of the Great Smoky Mountains at elevations above 6000 feet.

The headwater region of the French Broad, Nolichucky, and Pigeon Rivers is subject to heavy, high intensity rainstorms either of tropical hurricane origin or of summer thunderstorm characteristics. The susceptibility of the French Broad streams to these two types of storms seems to be even greater than that of the Hiwassee and Little Tennessee Rivers to the southwest. In general, the highest floods on the headwaters have occurred in the summer months, and the greatest floods on the lower reaches have resulted from winter and spring storms. Approximately two-thirds of the floods on the upper French Broad River and its tributaries occur in the months May-November with the greatest frequency in midsummer. Near Newport, floods occur most often in the December-April period but summer floods continue to be important.

The principal urban development on the river is at Asheville, North Carolina, and the adjoining Biltmore. Other cities in the watershed that are subject to flood damage are Marshall, on the French Broad River; Canton, Clyde, and Newport on the Pigeon River; the resort town of Gatlinburg, Tennessee, on the West Fork Little Pigeon River; and Sevierville, Tennessee, at the confluence of the Little Pigeon River and the West Fork.

Early power developments in the French Broad River watershed were all small, and none were effective for flood control. These included the Carolina Power and Light Company dams at Asheville and Marshall on the French Broad River, and near Waterville, North Carolina, on the Pigeon River; the Cascade Power Company dam on Little River; the East Tennessee Light and Power Company dam (purchased by TVA in 1945) on the Nolichucky River near Greeneville, Tennessee; the Champion Fibre Company water supply dam on West Fork Pigeon River above Canton, North Carolina; and a number of small mill dams. Between February 1942 and February 1943, TVA constructed Douglas Dam, situated 32.3 miles above the mouth of the river. The reservoir behind this dam provides a substantial degree of flood control on the lower French Broad River and on the Tennessee River.

Three great floods, all occurring in the past 50 years, have dominated the flood history of the upper French Broad, Pigeon, and Nolichucky Rivers. These were the flood of July 16, 1916, on the upper French Broad River and its eastern tributaries; the flood of May 22, 1901, on the Nolichucky River; and the flood of August 30, 1940, on the Piegon River and the western tributaries of the upper French Broad River. Great destructive floods also occurred in April 1791, June 1876, August 1928, and numerous other dates, but these three were outstanding.

The flood of July 16, 1916, was a maximum on the French Broad River from Rosman, North Carolina, to Bridgeport, Tennessee. Resulting from a storm of tropical hurricane origin, this great "freshet" was nearly 10 feet higher than any that have occurred since at Asheville and 5 feet higher than any in the past history of the city. Even upstream at Blantyre, where the river spreads over wide bottom lands, the flood was 4 feet above any other known flood. The heaviest precipitation during the storm fell along the Blue Ridge forming the eastern boundary of the upper French Broad River, and stages far surpassing any others in history occurred also on Little River, Mud Creek, and Cane Creek heading on this divide. Swannanoa River recorded its highest crest since the almost legendary flood of April 1791.

The second highest flood for which definite data are available occurred on the upper French Broad River in June 1876, although the flood of April 1791 may have exceeded it. The third highest flood of recent record was in August 1928, but higher stages are believed to have occurred in August 1796, May 1845, and August 1852.

Of the tributary streams entering the upper French Broad River from the west, Mills River and Hominy Creek experienced maximum known stages on August 30, 1940, while the greatest flood on Davidson River occurred in June 1876. Floods of August 1910, October 1918, August 1928, and August 13, 1940, were also of major importance on one or more of the tributaries above Asheville. On the North Fork of Swannanoa River, the flood of June 16, 1949, exceeded any since 1916.

On the lower French Broad River, the floods of March 1867 and February 1875 were outstanding. At Dandridge, which is below the inflow of both the Pigeon and Nolichucky Rivers, the two were equal in height. The floods of May 1901, February 1902, and July 1916 crested some 3 feet lower.

In the Nolichucky River watershed the July 1916 storm produced a maximum stage on the North Toe River and a flood in January 1927 was the highest on the South Toe River, but over the greater part of the Basin the flood of May 1901 exceeded all others. This flood was also a record in parts of the Watauga River Basin which lies just north of the upper Nolichucky. The second ranking flood over much of the upper Nolichucky River watershed occurred on August 13, 1940, while in the lower reaches the flood of March 1867 was nearly as high as in 1901.

On the upper Pigeon River and its West Fork, the flood of August 30, 1940, exceeded any previous known crest. There is some evidence to indicate that a flood in about 1810 may have been as high. On the East Fork the flood of June 1876 was a maximum, and this flood is second only to the crests of 1810 and 1940 at Canton. Floods of September 1893, which ranked second on the West Fork, and August 13, 1940, which ranked second on the East Fork, produced stages at Canton very nearly the same as in 1876. On the lower Pigeon River at Newport the winter floods again predominate and the crests of February 1902 and March 1867 are, respectively, first and second.

The greatest known flood on the Little Pigeon River and on its West Fork at Sevierville occurred in March 1875. A flood on April 1, 1896, was second highest on the Little Pigeon River at the gage downstream from the confluence with West Fork Little Pigeon River. On the West Fork the 1896 crest was approximately equal to that of 1875. A flood on April 2, 1920, ranked second on the Little Pigeon River above the influence of the West Fork.

Table 14 lists crest stages and discharges for the two highest known floods at several locations in the French Broad River watershed.

Very heavy damages result from floods in the French Broad River Basin. This is particularly true in the region above Asheville where even moderate overbank rises occurring in the summer months cause agricultural losses measured in hundreds of thousands of dollars. This region, subject to large losses in the past, has become even more vulnerable in recent years because of the development of a thriving truck crop industry. The wide, fertile botton lands along the main river and its tributaries in the Asheville flood plain produce bountiful high value crops of beans, cabbage, broccoli, and other vegetables which are destroyed when overflows occur.

In 1949 three floods occurring in June, July, and August caused damages totaling $1,219,000 in the

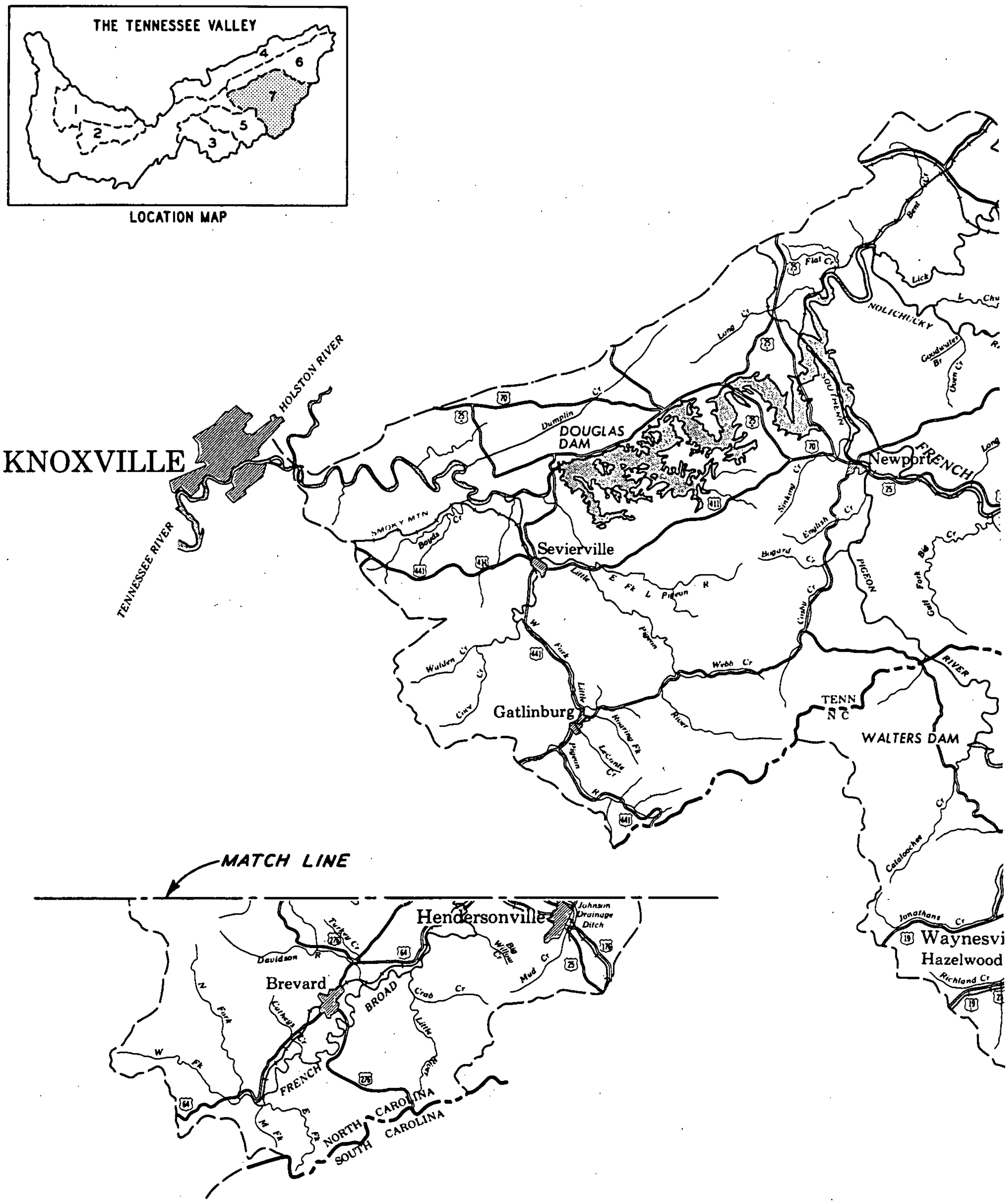
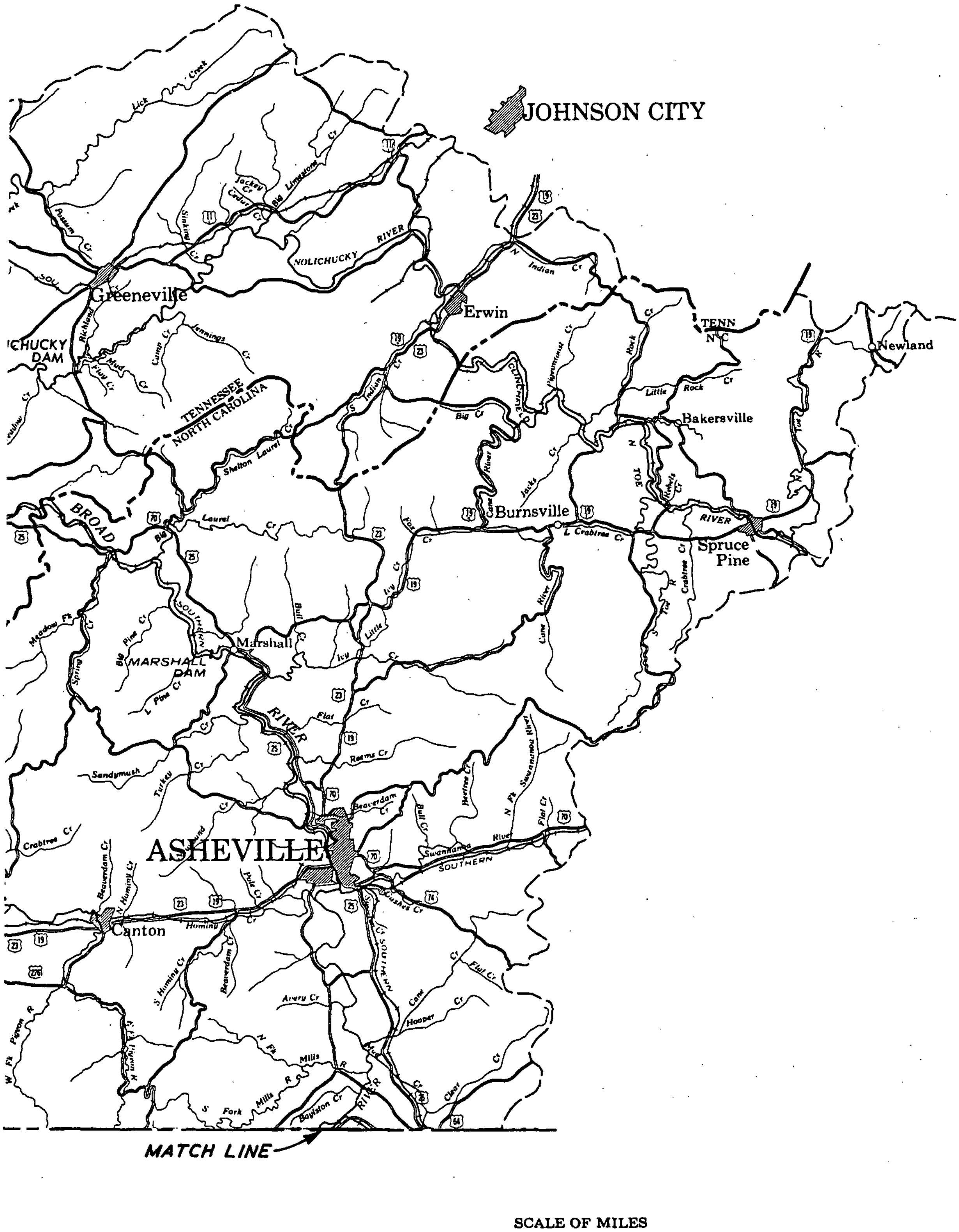


FIGURE 71.—Fre



ıad River Basin.

upper French Broad River area and $132,000 in the Swannanoa River and Hominy Creek Basins. Of the total loss, $943,000 was suffered by truck crop growers. The first flood, on June 16, destroyed $341,000 worth of beans and other truck crops that were nearly at the picking stage. Growers replanted immediately and, on July 12 to 19, another overflow caused a loss of $342,000 in truck crops, largely in the newly planted areas. Growers once more planted seed, hoping for a late crop, but on August 27-28 a third overflow occurred.

At Asheville and Biltmore, large floods on the French Broad and Swannanoa Rivers affect industries, business places, railroads, utilities, and homes and cause damage which, in 1916, totalled nearly two million dollars. Below Asheville, the city of Marshall, North Carolina, situated on a narrow strip of flat land on the river bank, is heavily damaged by floods of the magnitude of those in 1916 and 1940. Industrial losses are not confined to Asheville, however. A large paper and cellophane plant on Davidson River, an extensive rayon plant on Hominy Creek, and an important paper plant on Pigeon River are all vulnerable to flood damages.

On Nolichucky River and on the lower French Broad and Pigeon Rivers, losses are less concentrated and are suffered chiefly by highways, railroads, and agricultural interests.

TABLE 14.—*Maximum known floods at selected stations—French Broad River Basin.*

Stream and station	Drainage area, square miles	Maximum known floods: Date		Gage height, feet	Estimated discharge, cubic feet per second	Discharge per square mile, cubic feet per second
French Broad River:						
At Dandridge, Tennessee	4446	March	1867	25.2	140,000	31
		February	1875	25.2	140,000	31
At Asheville, North Carolina	945	July	1916	23.1	110,000	116
		June	1876	18	61,500	65
Nolichucky River:						
Near Morristown, Tennessee	1679	May	1901	26	85,000	51
		March	1867	25	76,000	45
At Embreeville, Tennessee	805	May	1901	24	120,000	149
		August	1940	18.57	82,500	102
Pigeon River:						
At Canton, North Carolina	133	August	1940	20.75	31,600	238
			1810	20	30,000	225
		June	1876	18	25,000	188
At Newport, Tennessee	666	February	1902	22	57,000	86
		March	1867	21.5	54,000	81
Swannanoa River:						
At Biltmore, North Carolina	130	April	1791	26	—	—
		July	1916	21	23,000	177
Mud Creek:						
At Naples, North Carolina	109	July	1916	21.5	40,000	367
		August	1910	15.5	19,000	175
Mills River:						
Near Mills River, North Carolina	66.7	August	1940	13.62	13,400	201
		August	1928	13.5	9,700	145
Little Pigeon River:						
At Sevierville, Tennessee	353	March	1875	18	55,000	156
		April	1896	16.8	46,000	130

Major tributary basins—flood summary

The flood of March 1867, which was a maximum on Tennessee River at Knoxville and Chattanooga, was also a maximum in the lower reaches of the Holston, French Broad, and Little Tennessee Rivers. The flood of February 1875, which was second highest on the Tennessee River, was second on the lower portions of Holston and French Broad Rivers and third on the Little Tennessee and Hiwassee Rivers. The great headwater floods of May 1901, July 1916, and August 1940, lacking support in the downstream tributaries, were of little importance by the time they reached Chattanooga.

The greatest floods in the western half of the Tennessee Valley, and north of the mountains in the eastern half, have all resulted from winter and spring storms. The greatest floods on the headwaters of the rivers rising on the Blue Ridge have occurred, generally, in the summer and fall, but in their lower reaches the winter and spring floods have been most important.

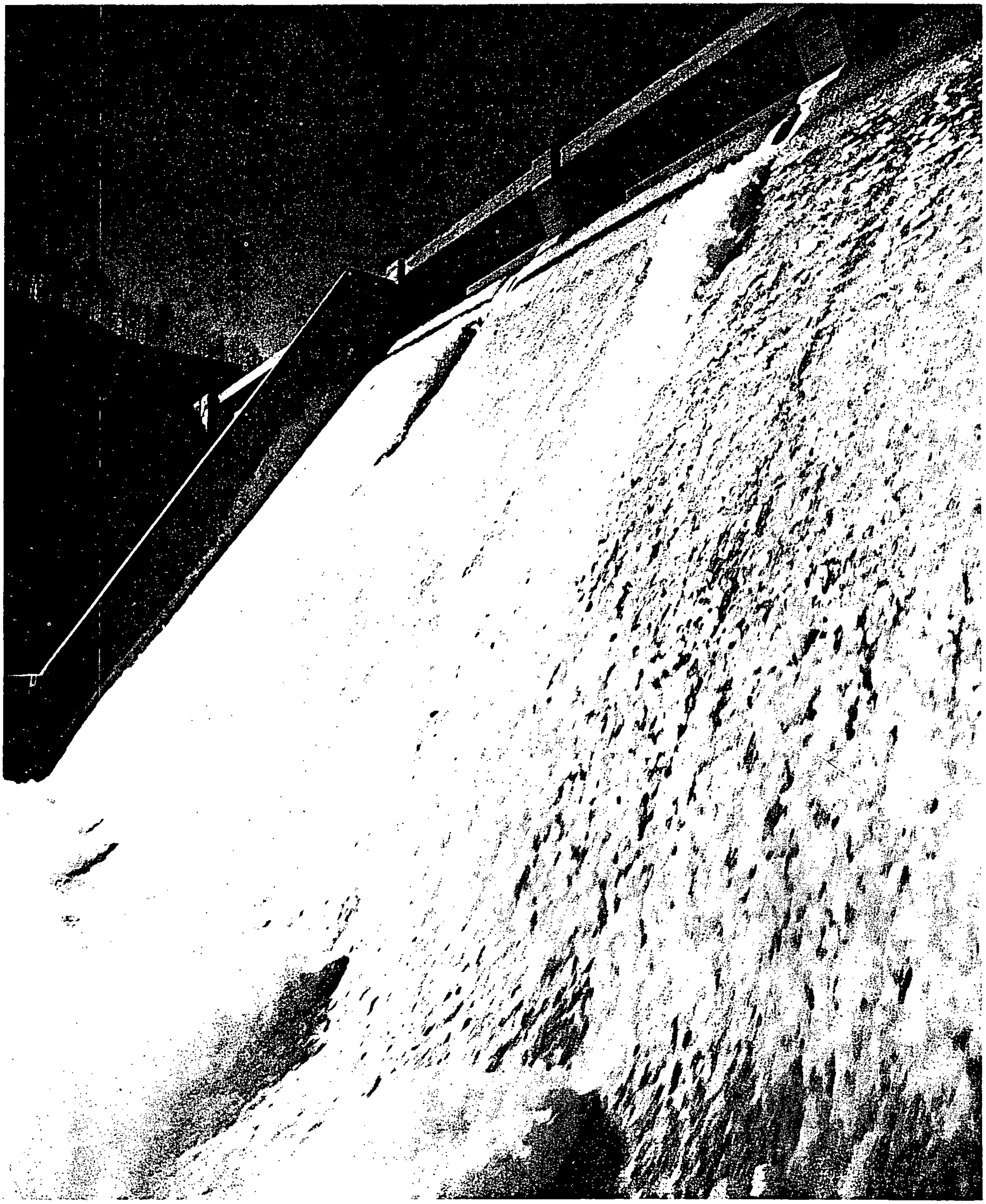


FIGURE 72.—*Norris Dam, a tributary flood control project, discharging stored flood waters in 1937 (This was the first dam built by TVA—it was completed in 1936).*

CHAPTER 5

DESIGN FLOOD FLOWS

Design flood flows are basic in the development of any plan for flood protection and this chapter covers their determination for use in developing the TVA system. First, the chapter describes the characteristics affecting flood runoff in the Tennessee Valley; it then discusses the methods TVA used to determine flood flows for design purposes. These discussions include definitions of the three types of floods—design, maximum probable, and maximum possible—considered in planning TVA projects.

CHARACTERISTICS AFFECTING FLOOD RUNOFF

Many complex, interrelated factors affect the frequency, height, and duration of floods that occur on a river. Basically, these factors are aspects of the climate of the basin or its physiographic situation, which are themselves interrelated.

Climatic factors

The elements of climate that are particularly important insofar as flood runoff is concerned are precipitation and temperature. These elements in turn affect other factors such as the type, extent, and seasonal variations in the vegetal cover.

Precipitation—Whether or not an important flood occurs on a stream as a result of a storm depends largely on (a) the amount of rain that falls on the watershed, (b) the area covered by the storm, and (c) the intensity or duration of the rainfall. Widespread winter storms of relatively low intensity but producing 6, 8, or 10 inches of rainfall in a period of several days will result in only moderate rises on small streams with a short time of concentration, but very high floods on the larger rivers. Cloudburst rains over an area of 10 or 20 square miles may cause devastating floods on small watercourses but a relatively minor rise on the larger streams.

Prior snowfall or rainfall are important contributing influences in flood occurrences. Snow on the watershed will melt during the storm rainfall and add directly to the runoff, while recent rainfall will fill soil storage and tend to increase the amount of storm rainfall that runs off during the storm.

In the Tennessee Valley, on all but the smallest streams the chances of a flood occurrence are greater in the winter season from November through March or April than in the rest of the year. Widespread cyclonic storms with heavy, persistent precipitation occur more frequently in the winter season. In the same months vegetation is dormant and ground surface conditions favor a high rate of runoff. Rainfall which may produce floods on sizable areas in the summer and fall months occurs generally in the eastern part of the Valley as a result of decadent tropical storms that move inland. Thunderstorms which produce floods on smaller streams are also more frequent in the summer months.

Temperature—The seasonal temperature variations influence the rate of evaporation from the soil and thus the degree to which the ground can store rainfall. In the summer, evaporation and evapo-transpiration losses from soil moisture are high, and much of the early part of the storm rainfall after a brief dry period is taken up by the soil. In the cold season the ground dries slowly after prior precipitation, and frozen ground may block infiltration. When no storage is available in the soil, or when storage is blocked by surface conditions, high stream flows may result from large rains even when the rate of rainfall is low.

Temperatures control the precipitation and subsequent melting of snow. As was previously pointed out, snowfall may be an important factor in flood runoff from the higher elevations of the Valley.

Vegetation—The climate of a region affects to a considerable degree the natural vegetation that grows on the watersheds of the region. Approximately 54 percent of the humid Tennessee Valley area is covered by forest, and in some of the mountainous tributary river basins the proportion of forest land ranges up to 84 percent. A good proportion of the land not forested is in pasture or is idle land covered by weeds, brush, and other natural vegetation.

Heavy, short duration summer rains may be absorbed completely by soils protected by dense forest. Studies on small areas in the Valley have shown substantial reductions in summer flood heights as a result of improving forest cover. A good pasture is nearly as effective as forest cover in inducing water to enter and flow through the soil rather than over it.

During moderate winter storms the effect of vegetation in checking flood runoff is reduced. Even without the interception by the leaves, however, vegetation in winter increases infiltration as compared with that for bare soil by retarding surface flow and by making the soil more pervious to infiltrating water.

When great storms strike a river basin, even a virgin forest cover will not prevent a high flood on the stream. The greatest known floods on some of the streams of the Tennessee Valley occurred in 1791, 1826, 1840, 1856, 1862, 1867, and other early years when forests, especially in the eastern area of the Valley, covered a high percentage of the watersheds.

Physiographic factors

Among the physiographic factors that have a significant effect on flood runoff from a stream basin are its location with respect to storm paths, the size, shape, and orientation of the basin, the stream pattern, and the topography. These various factors are very closely related to each other in their effect on flood runoff.

Location—The Tennessee River Basin, because of its location, is one of the wettest regions of the United States. The Gulf of Mexico and the Caribbean Sea, only a short distance to the south, provide major sources of moisture. No significant barriers lie between these sources of moisture and the Tennessee River Basin other than the boundary of the Basin itself. Prevailing winds from the south and west bring this moisture across the Basin.

The Basin is also within the range of hurricane storms that move across the Mississippi, Alabama, Georgia, South Carolina or North Carolina coastlines and bring enormous amounts of moisture into the Tennessee Valley region.

Size of basin—The areal extent of a stream basin has an important bearing on its flood experiences. Storm precipitation varies considerably with area. An intense thunderstorm covering 10 or 20 square miles may cause damaging floods on several small tributary areas but only a minor rise on the main stream into which they flow. In general, a stream with a small drainage basin will experience floods more frequently than one draining a large area and may rise to record flood heights in almost any month of the year. Sizeable floods on a large stream will be limited almost entirely to the months when widespread storms occur.

Shape of basin—The greatest flood at any location in a stream basin in the Valley region will result when the heaviest rainfall that may be expected to occur falls over the entire basin above the observation point within the time it takes for water from the most remote portion of the basin to reach the observation point. This time in which it takes flood waters to concentrate depends not only on the steepness of the basin and the slope of its water courses but also on the size and shape of the basin. If the basin is roughly semi-circular or fan-shaped, the contributions of each tributary will generally reach the observation point at about the same time, and the peak rate of discharge will approach the summation of the tributary peak flows. If the basin is long and narrow, the peak flows of the upstream tributaries may reach the observation point after the peak flows of the downstream tributaries have passed by. This lack of coincidence of peak flows results in a lower crest discharge from the long, narrow area than from the semi-circular or fan-shaped one.

This generalization may be modified by the stream pattern. Thus, the stream draining a long, narrow basin may have a trellis pattern with a main stem and with many short tributaries entering at right angles. Another long basin may be drained by two or more large streams flowing roughly parallel to each other and converging just above the observation point. All other conditions being equal, the former stream will have a lower flood crest than the latter.

The orientation of large river basins with respect to storm movements will also modify the effect of basin shape and stream pattern, particularly in the case of the long, narrow areas. A storm moving upstream over such a basin will produce a lower flood crest than one moving downstream.

Topography—The slope of the ground surface and stream channels in a basin influence the time it takes runoff from a storm to concentrate at any given point. In general, a basin with steep slopes produces its highest flood peaks from heavy storms of short duration. With low rates of rainfall, the water that runs off the steep slopes does not accumulate in sufficient volume to overtax the stream channels. A channel of a given cross section area and steep slope can, naturally, discharge water at a higher rate than a channel of the same area but flatter slope.

Watersheds with low relief and flat streambed slopes will generally have greater storage space in channels and flood plains which will modify the flood hydrograph. The effect of this is to remove and store water during the rising period of a flood, to lower the peak discharge rate, and to return the stored water during the falling stages of the stream. For a storm of given amount and intensity, a stream in a region of steep topography will rise more rapidly, crest for a shorter period, and fall more rapidly than a stream in a flatter region.

Where the ridges and mountains in a basin are as high as those in the southeastern portion of the Tennessee Valley, they have a significant effect on storm rainfall and total annual precipitation. During cyclonic storms, the lift imparted by the Blue Ridge and Great Smoky Mountains adds substantially to the amount of rainfall at the higher levels of these mountains as compared with areas not affected by them.

As a secondary effect, the valleys lying on the lee side of the high ridges generally receive less rainfall during a cyclonic storm than areas of the same elevation that are free of the effect of the mountain shielding.

Soil characteristics—The soils of a river basin are composed of various mixtures of silt, clay, sand, gravel, and organic matter. The movement of water within the soils depends to a large degree on the noncapillary pore spaces or channels in the soil and on the number of passageways resulting from decaying plant roots and other causes. The infiltration of rainfall through the soil surface into the soil depends on a large number of factors such as the porosity of the surface, the amount of water already in the soil, the rainfall drop size and intensity, the season of the year, air temperature, the presence of vegetation, and others. In general, a dry soil absorbs water more freely than a wet soil; a loose soil surface takes up water faster than a compacted one, a coarse sand or gravel soil transmits water more rapidly than a fine-grained silt or clay soil; a deep soil will take up and store more water than one which is supported near the surface by an impervious layer of hardpan or rock; and heavily vegetated land will absorb water more readily than a bare soil surface. All of these soil factors affect the runoff from a storm over the watershed.

Major tributary characteristics

All but two of the seven major tributaries of the Tennessee River are situated in the eastern half of the Basin. Characteristics that affect flood runoff in these tributary basins are generally similar, but there are some differences worthy of mention. The following discussions are largely directed toward pointing out these differences. Maps and detailed descriptions of these seven tributary basins appear in the preceding chapter.

Duck River—Flood-producing storms on the Duck River are limited almost entirely to the cyclonic disturbances of winter and early spring months. The basin is about 110 miles long and averages 32 miles in width, with the main stream flowing generally northwestward. Areas of heaviest rainfall during major storms usually cut across the basin in a northeast-southwest direction. The relief is comparatively low, being greatest in the eastern headwater region. Forests cover 53 percent of the basin. The major tributary, Buffalo River, enters the Duck River near the lower end of the basin.

Elk River—This basin is about 85 miles long with an average width of about 27 miles. The stream heads on the same region as the Duck River but flows generally southwestward. The basin is 34 percent forested, having the least forest cover of any of the major Tennessee River tributaries. Relief is generally low. Important floods on the main stream occur almost entirely in the winter and early spring. Areas of heavy rainfall during major storms often lie parallel to or along the river basin. Richland Creek, the major tributary, joins the Elk River 43 miles above the mouth and greatly influences the relationship of flood peaks above and below the confluence.

Hiwassee River—The basin of this river is similar to those of two other eastern tributaries, the Little Tennessee and French Broad Rivers, in that its upper portion lies in the rugged, high rainfall region of the Appalachian Mountains while its lower portion cuts across the Appalachian Valley subregion. The Hiwassee River heads on the Valley divide which forms the first barrier to moist air flowing northward and eastward from the Gulf of Mexico. A portion of this boundary faces to the southwest and a portion to the southeast so that heavy rains due to orographic lift are likely to fall on some part of the river headwaters regardless of storm direction. The basin is more fan-shaped than those of the Duck and Elk Rivers, being about 80 miles long by an average 34 miles wide. Ocoee River, the major tributary, joins the Hiwassee 34 miles above the mouth. Forests cover 64 percent of the watershed, most of this cover being in the mountainous portion. The river, flowing northwestward, is affected by general winter storms and by intense summer and fall storms.

Clinch River—The Clinch River Basin lies along the northeastern boundary of the Tennessee Valley in a region that is characterized by long narrow ridges and valleys, all bearing in a generally northeast-southwest direction. The basin, exclusive of the Emory River, is about 195 miles long by an average 23 miles wide, making it the longest and narrowest major basin in the Valley. The general direction of flow is to the southwest. Forests cover 48 percent of the main river basin and 84 percent of the tributary Emory River Basin. Flood-producing storms occur almost exclusively in the December through April period. Because of the shape of the basin and the shielding effect of the Appalachian Mountain area to the south, the frequency of major flooding on the Clinch River has been much less than on the other large tributaries. Snowfall in the headwater region has been a contributing factor in a number of the more important floods on the Clinch.

Little Tennessee River—There is little difference between this and the Hiwassee River Basin with respect to flood runoff characteristics. The basin is 76 percent forested. The upper reaches include some of the highest mountain peaks of the Blue Ridge and Great Smokies, ranging up to 6,500 feet above sea level. The watershed is some 80 miles long by 33 miles wide, almost identical in proportions to that of the Hiwassee River. The river, which flows generally northwestward, is affected by intense summer and fall storms in the mountains as well as the more general winter storms. Orographic influences are important in the mountain region. Two large tributaries, the

Tuckasegee and Nantahala Rivers, join the Little Tennessee in the upper reaches of Fontana Lake.

Holston River—A substantial portion of this basin is similar to that of the Clinch River, which it parallels. The exception is the Watauga River Basin which lies largely in the Appalachian Mountains and which has characteristics more like those of the Hiwassee, Little Tennessee, and French Broad Rivers. The total basin is 46 percent forested, although the Watauga River portion has forest on 57 percent. Total length of the basin is about 160 miles; average width is 24 miles. The upper portion is generally fan-shaped with the North Fork and the South Fork (which includes the Watauga Basin) joining just below Kingsport. The basin is largely shielded by the Appalachian Mountains from the summer and fall hurricane storms and its principal floods result from winter cyclonic disturbances. The Watauga River portion experiences severe floods in all seasons of the year.

French Broad River—This basin above Douglas Lake is drained by the French Broad River itself and two large tributaries, the Nolichucky and Pigeon Rivers. The confluence of these three basins forms one of the outstanding "fan-shaped" areas in the Teneseee Valley. The total basin is about 95 miles long and averages 54 miles in width. Some 60 percent of the basin lies in the Appalachian Mountains where elevations range up to 6,684 feet on Mount Mitchell. The highest divide in the French Broad Basin is that along the western boundary. This is an important factor with respect to storms moving in from the east, causing them to deposit heavy rainfall on the watersheds of the western tributaries of the river. About 57 percent of the watershed is forested. The general direction of flow is to the north and west. The French Broad headwater area seems to be more susceptible to the decadent hurricane type of storm than that of any other major basin. More than half of the floods of record in the upper French Broad River Basin have occurred in the summer and fall months. Some of the greatest floods of record have occurred in July and August.

METHODS OF ESTIMATING GREAT FLOODS

The development of any plan for flood protection and the design of all hydraulic structures require the adoption of a design flood flow. The risks involved and the type of structure under consideration, whether it is a concrete dam, an earth dam, a levee protecting a large city or one protecting farmland from crop flooding, will influence the selection of the design flood magnitude. The magnitude of the flow will also affect the proportions of the structure and, consequently, its economic feasibility.

The outlet works at most of the TVA-built projects were planned to function normally in the maximum probable flood (the greatest flood which may reasonably be expected to occur at the site) which was based principally on the greatest observed floods in the Tennessee River Basin and in similar areas in the eastern United States. Adequate freeboard was provided above the level that would be reached at the dam in such a flood.

It would be impracticable to list all sources of information or to review all computation methods utilized by TVA to determine maximum floods for design purposes. Only those methods are described which were extensively used and which would be used again under similar circumstances.

Floods considered in planning TVA projects

To avoid a misunderstanding in the use of terms, three types of floods considered in planning the TVA projects are defined in the following paragraphs.

Design flood—A flood adopted for use in determining the hydraulic capacity or proportions of a structure, such as the outlet works of a dam, the height of a dam or levee, or the maximum water level in a reservoir is designated the *Design Flood.* The magnitude of the design flood depends largely on judgment and the type of structure. *It may be any magnitude considered appropriate for the particular use.*

Maximum probable flood—This flood is the greatest that may reasonably be expected, taking into account all pertinent conditions of location, meteorology, hydrology, and terrain. Its magnitude may be judged from flood rates observed over a broad region having a large number of watersheds of various sizes but with similar hydro-meteorological characteristics. Because it is based on actual flood observations, there is a reasonably good chance of its occurrence, and it may occur in any year. Such a flood would very likely be less than the maximum possible flood. The frequency of this flood is not susceptible of determination. Most of the TVA projects are designed for a flood of this magnitude as modified by upstream storage in existence at the time of construction.

Maximum possible flood—This flood is the greatest that could occur assuming complete coincidence of all factors that would produce heaviest rainfall and maximum runoff. It would result from the maximum possible rainfall as determined by the transposition to this area of maximum observed rainfall adjusted for differences in maximum observed moisture charge and wind velocity between this area and the area where the storm occurred. The frequency of this flood is not susceptible of determination, but its occurrence would be highly improbable.

Uses of design flood

The principal use of the design flood is to determine the discharge capacity of the spillway and outlet works required at dams. At some projects, especially those which were built before upstream flood storage reservoirs were authorized, it was necessary to know only the peak discharge rate. In the other cases where there was a substantial flood storage space available at the project itself or at upstream projects, this peak rate was reduced in accordance with the degree of control provided. To determine this reduction the volume as well as the peak rate of the design flood must be known. The discharge capacity of the outlet works when the water level is at its maximum must be at least equal to the adopted design flood peak flow after it has been modified by storage. It may be greater than this rate because of other considerations, as at Kentucky Dam, where it is necessary to provide spillway capacity to maintain minimum headwater level at times of high inflow in order to preserve storage space for later use. In this case the discharge capacity required at low headwater elevations resulted in a capacity at high levels greater than the maximum probable flood without upstream regulation.

The amount of freeboard between maximum water level of the design flood and top of dam depends on the type of structure. Usually with earth dams there is an allowance of from 10 to 15 feet, and with concrete dams from 5 to 7 feet.

In the design of storage dams the maximum probable flood was used to determine the capacity of the outlet works, and also the height of the dam and the reservoir storage to be provided for control of the flood. These three factors — outlet capacity, storage, and height of dam — are closely interrelated, and variations in one will affect the other two.

Where the purpose of a project is to protect life and valuable property, the structures usually are designed for the maximum probable flood, but often also with an investigation of the effect of the maximum possible flood. For example, the height of a dam may be determined for a maximum probable flood with freeboard added, and a calculation then made to determine if the dam would be overtopped by the maximum possible flood.

In the case of flood control for protection of agricultural land only, a design flood smaller than either the maximum possible or maximum probable flood may be adopted. Here buildings, particularly for human habitation, are constructed outside the flooded area, and loss of life is usually not involved. An occasional flooding of farmland, or even the occasional loss of a crop, would not be a catastrophe. It might be no more serious to the farmer than such other natural forces as droughts and windstorms. The benefits, therefore, would not justify the cost of protection against extremely high floods; and lower flood flows, as determined by the economics of the situation, would be used in the design of channel improvements and levee heights for farm protection.

Peak rate of discharge

Three basic characteristics of a flood discharge hydrograph are (1) peak rate, (2) total runoff volume, and (3) successive variations in rate of flow with time. Another important characteristic is the probable date or season of occurrence. Although these characteristics are closely related, they may be determined more or less independently.

In table 15 are listed the highest floods in the Tenneseee River Basin at long-established stream gaging stations. The coefficient "C" is a convenient

TABLE 15.—*Crest stages and discharges of maximum known floods at selected locations in the Tennessee Valley.*

Location	Date	Stage, feet[1]	Discharge, cubic feet per second	Drainage area, square miles	Coefficient, $C=\frac{cfs}{\sqrt{sq.\ mi.}}$
Tennessee River at Knoxville	March 8, 1867	45.0	290,000	8,913	3,080
Tennessee River at Loudon	March 8, 1867	47.8	403,000	12,220	3,650
Tennessee River at Chattanooga	March 11, 1867	57.9	459,000	21,400	3,140
Tennessee River at Florence	March 19, 1897	32.5	470,000	30,810	2,680
Tennessee River at Johnsonville	March 24, 1897	48.5	475,000	38,530	2,420
French Broad River at Asheville	July 1916	23.1	110,000	945	3,580
French Broad River at Dandridge	March 7, 1867	25.2	150,000	4,446	2,250
South Fork Holston River at Kingsport	May 1901	23	100,000	1,931	2,280
Holston River near Rogersville	March 7, 1867	35.4	150,000	3,035	2,730
Little Tennessee River at McGhee	March 1867	39.5	150,000	2,443	3,040
Hiwassee River at Reliance	September 1898	31	90,000	1,223	2,580
Clinch River at Clinton	March 31, 1886	41.5	114,000	3,056	2,060
Emory River at Harriman	March 23, 1929	61.1	180,000	798	6,370
Elk River at Prospect	February 14, 1948	38.17	100,000	1,784	2,370
Duck River at Columbia	February 14, 1948	51.75	61,100	1,208	1,760
South Fork Holston River at Bluff City	March 1867	21	46,000	813	1,610
Watauga River at Elizabethton	May 1901	21.5	90,000	692	3,420
North Fork Holston River at Mendota	1862	19.1	41,000	493	1,850

1. Stage on present gage.

factor for comparing the relative magnitude of the floods. In figures 25 (page 22) and 73 are shown the yearly and seasonal occurrences of past floods at two stations, Chattanooga on the Tennessee River as representative of a location where great Valley-wide floods are confined to a part of the year, and at Kingsport on the South Fork Holston River as representative of a location where floods occur throughout the year. Heights of the floods occurring before systematic records were commenced were determined using the high-water marks located by field surveys. Flood heights during the period of gage record were taken from publications of the United States Weather Bureau or the United States Geological Survey. Peak discharges were determined from rating curves, computed flow profiles, or were taken from published Water Supply Papers of the United States Geological Survey. Floods occurring since the closure of upstream dams were computed crests which would have occurred without regulation.

Although the period of record differs at each of the stations shown in table 15, enough is known about the flood history of the streams in the Tennessee River Basin to conclude that no higher floods have occurred on the larger drainage areas in approximately 150 years. For example, there are records of large floods at Chattanooga on the Tennessee River in 1826 and 1847, and at Asheville on the French Broad River in 1791, but these were not higher than those listed in table 15. It may be concluded, therefore, that the probability of recurrence of the highest recorded flood is about once in 100 to 150 years on the average.

If flood records at a single gaging station are for a short period, it is unlikely that the greatest flood the stream is capable of producing will be included in those records. As the length of record increases, the chance of such a flood being included in the record becomes greater. But it would take many years, perhaps several hundred, before it could be said with confidence that the maximum flood capabilities of the stream had been experienced.

It seems a reasonable assumption, however, that on some streams the maximum flood of which those streams are capable of producing already has occurred. Under this assumption floods on many streams were compared with the additional assumption that the maximum capabilities of each stream would be equal to the maximum flood of the entire group of streams.

To make the comparison, maximum flood crest discharges on selected areas in the Eastern United States were collected from available published sources. The drainage areas in the Ohio River Basin and in streams draining the North Atlantic slope, being most similar to the Tennessee Basin and subject to the same general type of storms, provided the most useful data. As an added precaution that the adopted maximum probable flood would be based on the highest observed floods, data on streams in the St. Lawrence, upper Mississippi, Missouri, and lower Mississippi River Basins were also collected, even though climate and basin characteristics there differ from the Tennessee River Basin. The comparison of floods in all selected areas was made by dividing the

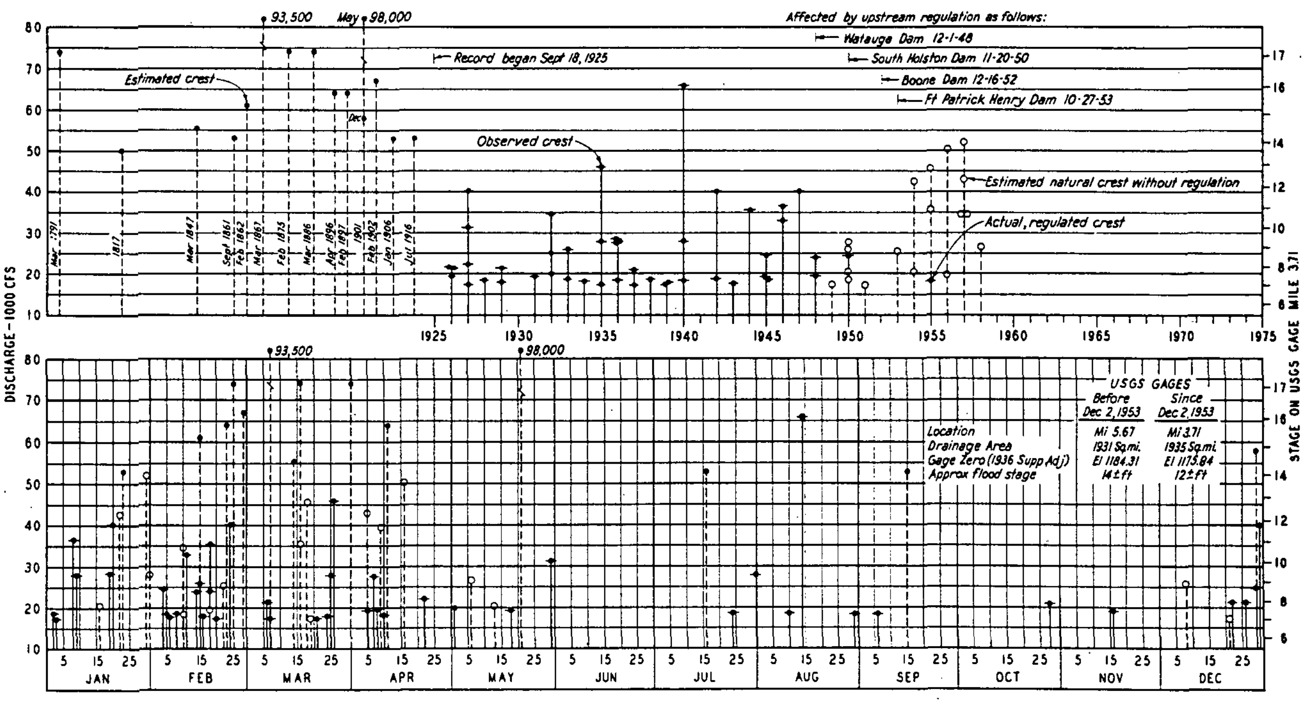


FIGURE 73.—*Yearly and seasonal occurrences of floods—South Fork Holston River at Kingsport, Tenn.*

flood flow in cubic feet per second by the drainage area in square miles and by plotting that rate against drainage areas in square miles on logarithmic cross-section paper (fig. 74). This is a standard procedure for comparing floods on drainage areas of various sizes. Flood rates for some of the outstanding storms were shown by different symbols as indicated on the chart.

The slope of the mass of the points on the chart shows in a general way that the flood rate in cubic feet per second per square mile varies inversely with the square root of the drainage area. If q is the flood rate in cfs per square mile, A is the drainage area in square miles, and C is a coefficient, then $q=C/A^{1/2}$. A line having a slope of ½ drawn through the uppermost points would determine the maximum flood rates from any area, but in applying values from such a line consideration must also be given to (1) the accuracy of those uppermost points, (2) whether or not the storm could have occurred over the area under study, and (3) whether the physical characteristics of the basin under study are similar to those where and when the flood actually occurred.

Flood rates equivalent to 6,000 divided by the square root of the drainage area are supported by only four points in the area range of 700 to 4,000 square miles. Rates equivalent to 5,000 divided by the square root of the drainage area are supported by many points in the range of 500 to 30,000 square miles.

The search for flood rates for areas of less than 500 square miles was limited to those which were conveniently available. A more intense search to locate more rates in this range did not seem justified. However, those floods that were found for smaller areas indicate maximum flood rates in excess of 6,000 divided by the square root of the drainage area.

Obviously, fewer flood rates are available for large areas because of the smaller number of large streams. Consequently, the chances that the sum of the records for large streams already include the maximum probable flood are less than for smaller areas.

Consideration of the maximum observed floods in the Tennessee Valley and other comparable sections has led to the conclusion that, with certain exceptions, a suitable maximum probable flood rate in the Tennessee Valley is 5,000 divided by the square root of the drainage area. In the steep, mountainous sections and where the shape of the basin, the shallow depth or soil, or the chance of occurrence of excessive rainstorms is conducive to heavy concentration of runoff, a greater maximum probable flood should be adopted. A coefficient of more than 6,000 has already been experienced during the flood of March 1929 in the Emory River Basin and during the flood of March 1955 on Shoal Creek at Iron City, Tennessee. A coefficient of 6,000 was adopted for projects in mountainous sections such as at Fontana, Hiwassee, and Watauga, and for the small units of the upper French Broad River flood protection scheme.

In other portions of the Tennessee Valley the basin characteristics and the flood history indicate that a coefficient lower than 5,000 may be applicable. An example in the extreme northeastern portion is the North and Middle Forks of the Holston River. These basins are also long and narrow and, therefore, do not produce a rapid concentration of runoff. Another example of where a coefficient lower than 5,000 might be applicable is on tributaries in the western portion of the Basin, such as the Elk and Duck Rivers. Here, although the mean annual rainfall is near the average for the whole Valley, the long, sinuous channels and wide, relatively flat floodplains are indicative of a relatively large channel storage, thus resulting in lower peak discharges. No flood storage projects have been built by TVA on the North and Middle Forks of the Holston River nor on the Elk or Duck Rivers.

This use of the relation between peak discharge and drainage area was concurred in by a Board of Consulting Engineers[1] who submitted a report to TVA in May 1936 on "Certain Flood Problems in the Tennessee Valley." They advised the use of a peak rate of flow determined from the formula $Q=C\sqrt{A}$, in which Q is the maximum rate of flow in cubic feet per second, C a coefficient equal to 5,000, and A the drainage area in square miles. The Board stated that this equation

> . . . may be used for general application to drainage areas of more than 500 square miles in the Tennessee River Valley above Chattanooga, although in some cases it may be necessary to materially increase or decrease the values so determined in order to allow for special local conditions.

With respect to the Valley below Chattanooga, the Board was not specific. They recognized, however, that lower flood rates had been experienced there, but stipulated a flood rate only at Johnsonville where the drainage area is 38,500 square miles. This stipulated rate was 20 cubic feet per second per square mile. They added, however, that if the proposed structures consisted of earth embankments that this value should be increased by 20 percent.

Table 16 lists the natural unregulated maximum probable flood discharge and coefficient on the basis of the consulting board's recommendations. The values given for projects on the Tennessee River below Chattanooga do not include the 20 percent factor of safety for earth embankments. The interpretation of the Board's recommendation here is that the lower value is the maximum probable flood.

The table also gives the maximum probable flood as it would be regulated by upstream storage. These flows were determined at some locations, as at Chattanooga and the tributary dams, from a detailed

1. Harrison P. Eddy, Ivan E. Houk, Gerard H. Mathes, and Daniel W. Mead.

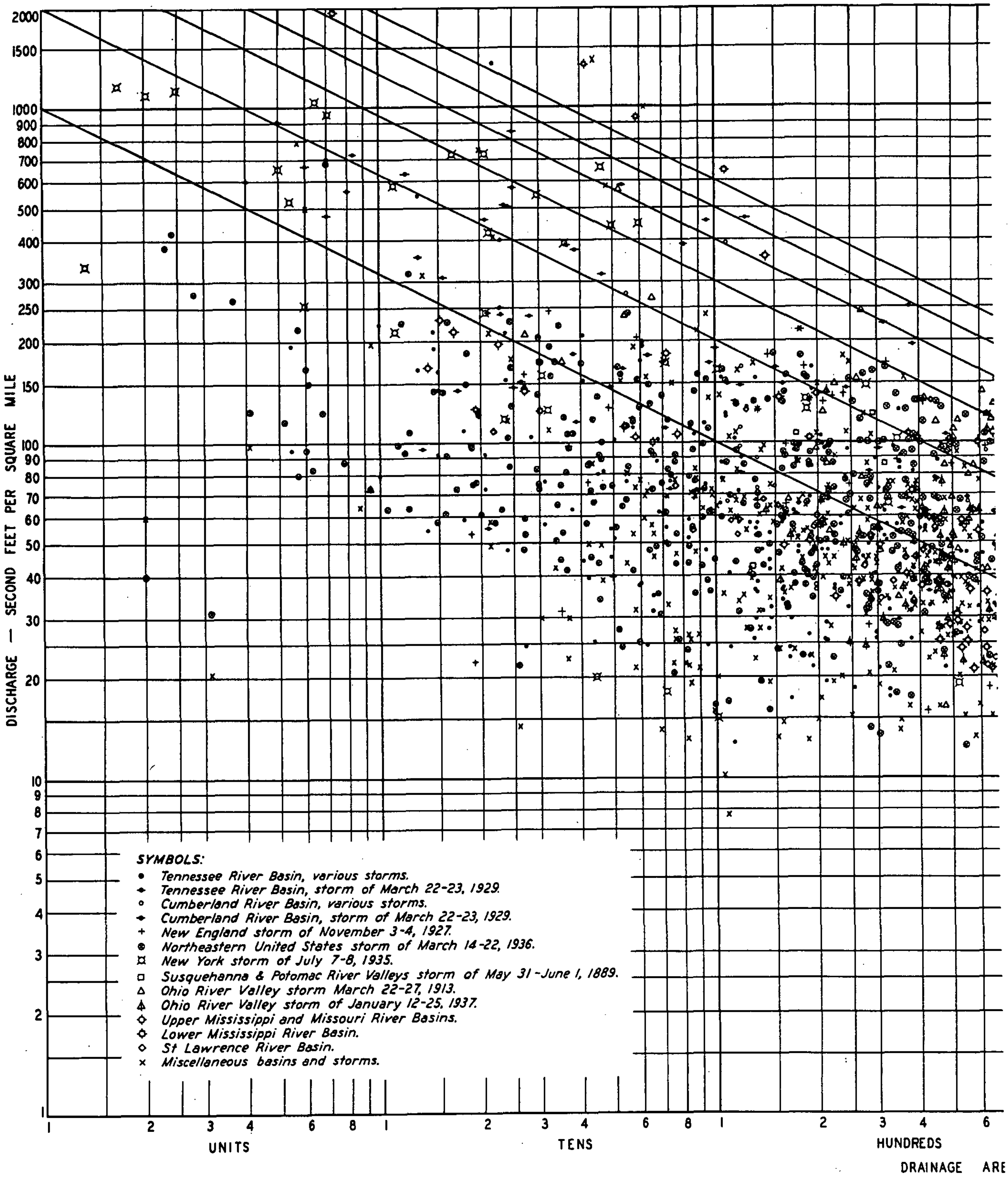
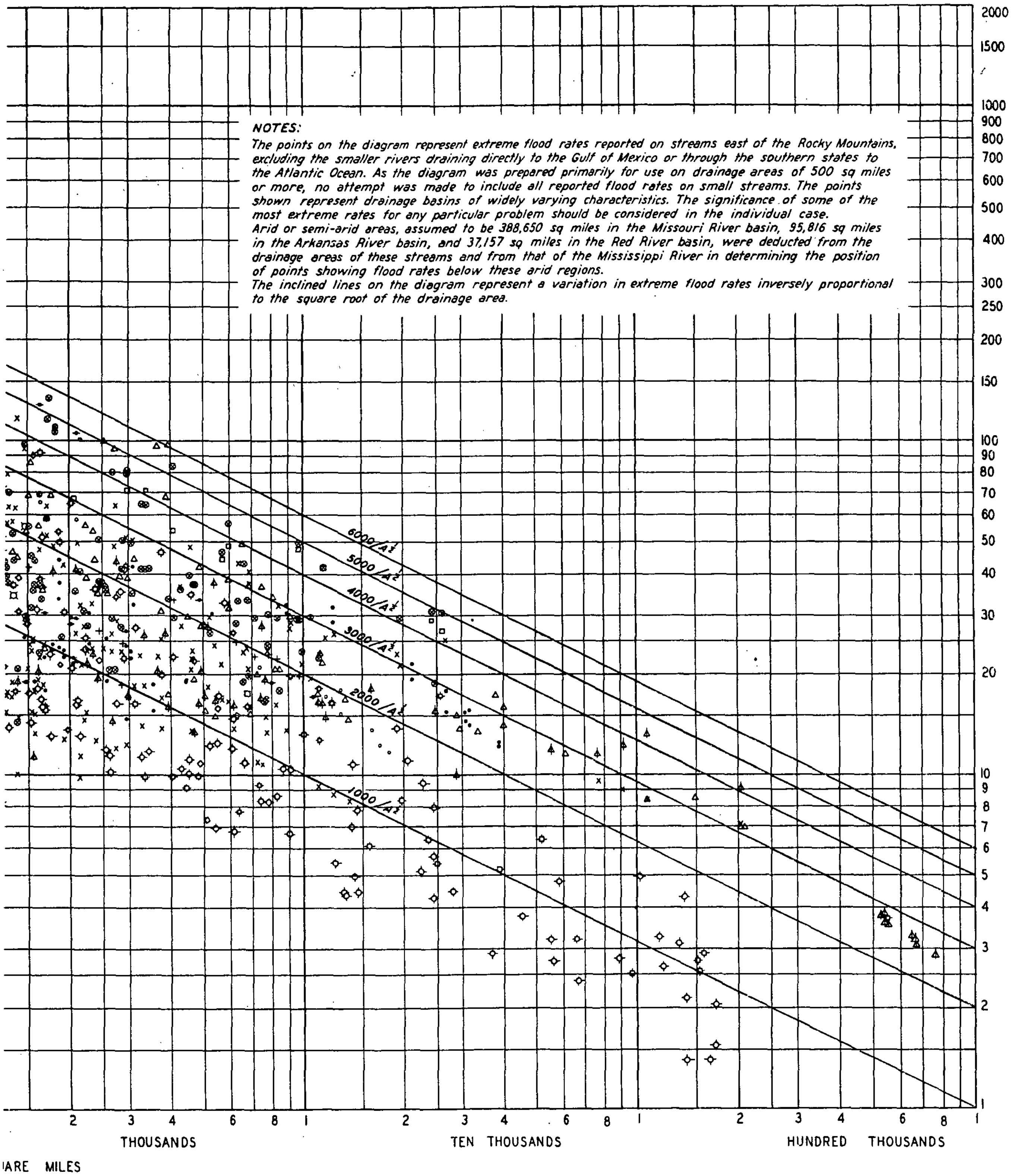


FIGURE 74.—*Extreme flood disc*



:—*Eastern United States.*

day-by-day routing of the flood. At other locations they were determined by analogy with the results of the detailed routings of many floods—pre-reservoir, post-reservoir, and hypothetical. These regulated flows are not necessarily the values used in the design of the spillways. When the dams were constructed, the regulated design flood depended on the upstream flood storage assured at that time and not on an amount which might be available at some future time.

For comparison with the unregulated and regulated maximum probable floods, spillway, sluice, and turbine capacity at each project is also given in table 16. Discharge capacity at the maximum elevation is not always the controlling factor in the determination of the length of spillway. At the main Tennessee River dams it is important that the limited storage space be held empty until the peak of the flood. This can be accomplished only with large spillway discharge capacity at the minimum elevation and may result in an apparent excessive capacity at the maximum elevation.

In addition to dams, table 16 lists the adopted maximum probable flood at several critical flood locations in the Tennessee Valley. At Chattanooga the adopted flood flow was 730,000 cubic feet per second, corresponding to a stage of 77.2 feet. This is 60 percent greater than the maximum known flood of March 1867 when the stage was 57.9 feet and the corresponding flow was 459,000 cubic feet per second.

Storm rainfall

A number of the important storms that have occurred over the Tennessee Valley are discussed in chapter 2. Isohyetal maps and area-depth curves are given in that chapter for several of these storms.

Some of the heaviest storms experienced in the Valley have been limited in extent to only a few of the tributary basins. The storms of May 1901, July 1916, and August 1940, for example, occurred largely over the southeastern tributaries; those of March 1826 and January 1918 over the northeastern section; that of March 1929 over the north central area; and those of March 1897, March 1902, February 1948, and March 1955 over the western portion.

The storms that have produced the greatest floods on the Tennessee River at Chattanooga have been widespread storms which deposited heavy rainfall over all the major tributaries. Table 17 shows the estimated storm rainfall over the five major

TABLE 16.—*Maximum probable floods and discharge*

			Maximum probable flood			
			Unregulated			
Project	Stream	Drainage area sq. mi.	Discharge cfs	Coefficient C[1]	Volume ins.[2]	Regulated discharge cfs
Kentucky	Tennessee River	40,200	804,000	4,010		804,000
Pickwick	" "	32,820	780,000	4,300		735,000
Wilson	" "	30,750	772,000	4,400		697,000
Wheeler	" "	29,590	768,000	4,460		675,000
Guntersville	" "	24,450	746,000	4,770		564,000
Hales Bar	" "	21,790	733,000	4,970		496,000
Chickamauga	" "	20,790	721,000	5,000		478,000
Watts Bar	" "	17,310	658,000	5,000		436,000
Fort Loudoun	" "	9,550	489,000	5,000		360,000
Apalachia	Hiwassee River	1,018	192,000	6,000		151,000
Hiwassee	" "	968	187,000	6,000	8	151,000
Chatuge	" "	189	82,500	6,000	10	40,000
Ocoee No. 1	Ocoee River	595	146,000	6,000		110,000
Ocoee No. 2	" "	516	136,000	6,000		111,000
Ocoee No. 3	" "	496	133,000	6,000	8	110,000
Blue Ridge	" "	232	91,400	6,000		
Nottely	Nottely River	214	87.700	6,000	10	57,000
Norris	Clinch River	2.912	307,000	5,700	11.1	38,000
Fontana	Little Tennessee River	1,571	239,000	6,000	10	188,000
Douglas	French Broad River	4,541	337,000	5,000	9	323,000
Cherokee	Holston River	3,428	294,000	5,000	8	142,000
South Holston	South Fork Holston River	703	133,000	5,000	8.5	76,000
Watauga	Watauga River	468	130,000	6,000	9.0	33,000
Boone	South Fork Holston River	1.840	215.000	5,000	8	137,000
Fort Patrick Henry	South Fork Holston River	1,903	218,000	5,000	8	145,000
Other critical points:						
Chattanooga	Tennessee River	21.400	730,000	5,000	12.9	486,000
Knoxville	" "	8.934	473,600	5.000		348,000
Asheville	French Broad River	945	154.000	5.000	12	154,000
Kingsport	South Fork Holston River	1,931	220,000	6,000	8.5	143,000
Elizabethton	Watauga River	692	158,000	6,000	9	92,000

[1] $C = Q / \sqrt{\text{area in square miles}}$.
[2] Inches depth over drainage area.
[3] Top of flashboard.
[4] Low level outlet.
[5] Spillway crest.

tributary basins above Chattanooga that caused the first, third, and fourth ranking floods at that city and the rainfall that would have caused the second ranking flood without TVA control. Average rainfall amounts for the 1867, 1875, and 1886 storms in this table are based on isohyetal maps drawn in connection with a study of maximum precipitation in the Tennessee Valley made by the TVA and the U. S. Weather Bureau.

In the 1867 storm, about 9 inches of the total fell in 4 days and nearly half of the total occurred in one day. In 1875 there was a total rainfall of 10 inches spread over 9 days of which the middle 3 days was without rain. The rainfall shown in the table fell in 3 days, with a maximum day of 3.4 inches. In March-April 1886 a total of 9.6 inches fell in 12 days. In this period there were also 3 days virtually without rain. The 6.6-inch amount shown fell in 6 days. In January-February 1957 there was a 21-day rainy period total of 12.2 inches in the area above Chattanooga. Some 3.3 inches of this fell between February 3 and 10, after the Chattanooga natural flood crest would have been reached. Only one-quarter inch fell on the 3 days prior to the main flood-producing rain of 7.3 inches.

Flood volume

In the case of projects having no appreciable upstream or self-contained flood storage, it is necessary to know only the peak discharge in order to determine the required capacity of outlet works. Where flood storage is provided, either at the immediate project or farther upstream, the capacity of the outlet works usually is based on the reduced crest flow which would result if that storage were utilized in the maximum probable flood. This, of course, requires a knowledge of the total volume of runoff in such a flood. This volume must be based on reasonable assumptions as to probability of occurrence and as to the relation of volume to peak discharge.

One method of determining the flood volume is to compute the relation between peak discharge and runoff volume in actual floods in the area under study. If the ratio of volume to peak is fairly constant for many floods, the application of that ratio to the adopted peak discharge rate of the maximum probable flood would result in a reasonable flood volume based on actual occurrences. This method accounts for all the physical characteristics of the stream, at least within the limits of observation, which

capacities of TVA projects and other critical points.

Spillway discharge capacity: At maximum reservoir level used for design		Spillway discharge capacity: At gate top level		Sluice discharge capacity (top of gate)	Total discharge capacity (top of gate)	Approximate turbine discharge capacity		
Elevation	cfs	Elevation	cfs	cfs	cfs	cfs	Units	Project
380.0	1,232,000	375.0	1,050,000	None	1,050,000	50,000	5	Kentucky
430.0	900,000	418.0	650,000	"	650,000	81,000	6	Pickwick
507.88	796,000	507.88	796,000	"	796,000	73,000	18	Wilson
558.3	665,000	556.3	542,000	"	542,000	80,000	8	Wheeler
605.0	650,000	595.44	478,000	"	478,000	44,000	4	Guntersville
648.0	506,000	635.0	224,000	"	224,000	44,000	16	Hales Bar
701.0	685,000	685.44	470,000	"	470,000	35,000	4	Chickamauga
745.0	560,000	745.0	745,000	"	745,000	40,000	5	Watts Bar
815.0	390,000	815.0	390,000	"	390,000	26,000	4	Fort Loudoun
1282.0	156,600	1280.0	136,000	"	136,000	2,900	2	Apalachia
1532.0	127,000	1526.5	90,000	22,000	110,840	7,300	2	Hiwassee
1933.8	39,300	1928.0	11,700	Use restricted	11,700	1,300	1	Chatuge
841.5	42,500	837.65[3]	19,000	Unserviceable	19,000[3]	2,600	5	Ocoee No. 1
		1115.2	0	Unserviceable		1,000	2	Ocoee No. 2
1438.0	117,000	1435.0	95,000	2,700	95,000	1,300	1	Ocoee No. 3
1700.0	55,000	1691.0	26,700	1,100	27,800	2,200	1	Blue Ridge
1787.4	49,400	1780.0	11,600	None	11,600	1,600	1	Nottely
1052.0	210,000	1034	55,000	37,920	92,920	8,100	2	Norris
1720.0	158,000	1710.0	104,000	25,100	134,300	7,200	3	Fontana
1002.0	313,000	1002.0	313,000	5,200[4] 29,000	334,700	15,100	4	Douglas
1075.0	256,000	1075.0	256,000	30,000	283,400	14,000	4	Cherokee
1755.0	62,000	1742.0[5]	0	10,700	10,700	2,400	1	South Holston
1981.5	11,500	1975.0[5]	0	11,300	10,700	2,700	2	Watauga
1385.0	137,000	1385.0	137,000	3,830	140,280	9,800	3	Boone
1263.0	141,000	1263.0	141,000	None	141,000	8,300	2	Fort Patrick Henry

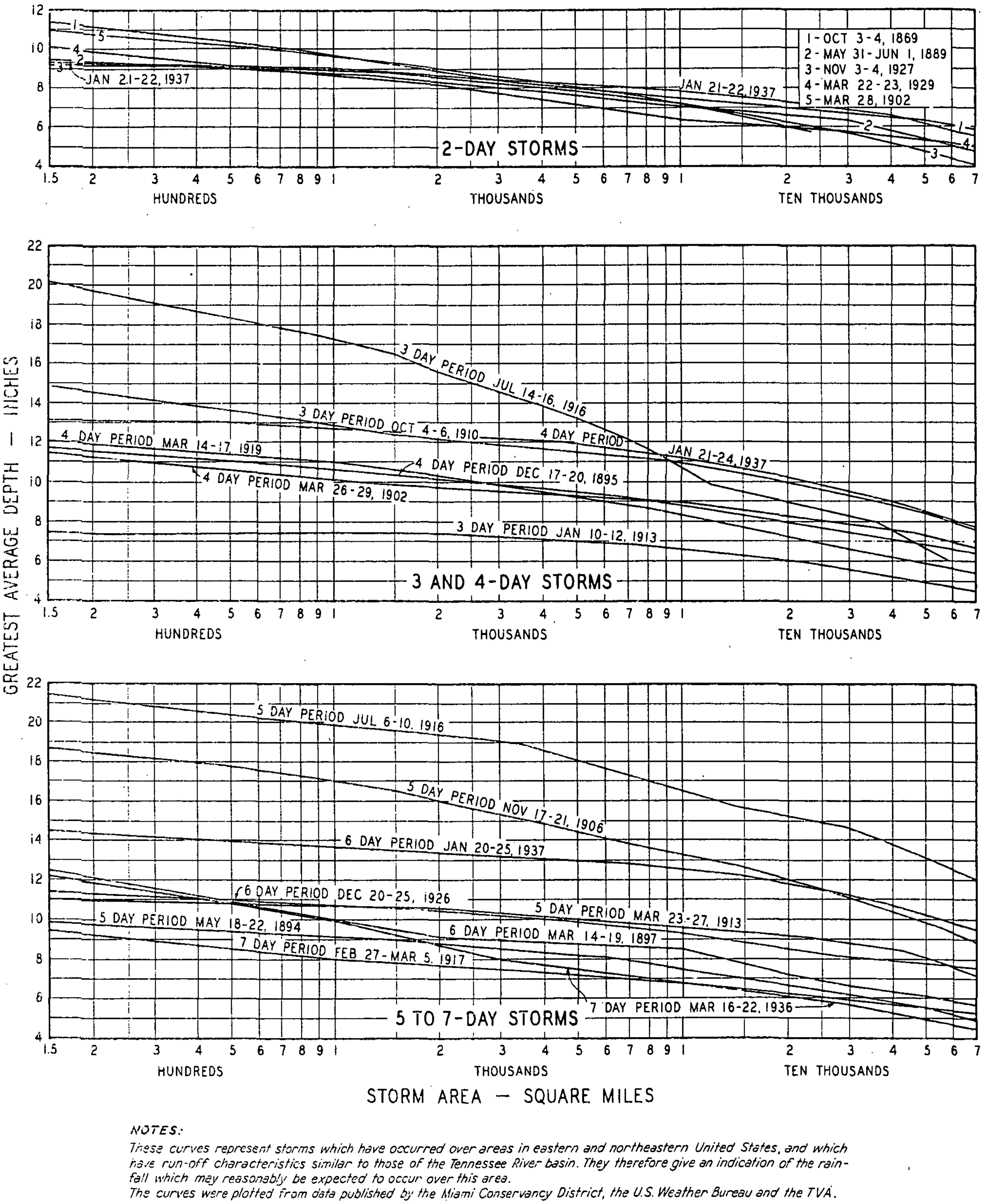


FIGURE 75.—*Storm rainfall—Eastern United States.*

affect the flood hydrograph. Because of overlapping storm and flood periods and differences in distribution and intensity of storms, however, the ratio of volume to peak varies widely in the Tennessee Valley.

The selected flood volume for the maximum probable flood was also related to the amount of rainfall in great storms that have occurred in and near the valley. Figure 75 shows the amount of rainfall in great storms that have occurred in the eastern United States over areas having characteristics similar to those of the Tennessee River Basin. Rainfall amounts were determined from isohyetal maps of the individual storms. Even though these storms did not occur in positions which would produce that rainfall over a particular basin, they might so occur. The amount of runoff from these storms may be estimated by applying to the rainfall a runoff percentage based on observed quantities of rainfall and runoff, or by subtracting a uniform water loss rate from the rainfall, also based on observed rainfall and runoff. The first method would be more generally applicable to large areas, say above 10,000 square miles, and the second method to smaller areas. Variations in distribution of rainfall over large areas make it impracticable to apply a uniform loss rate to the rainfall.

The adopted volumes of maximum probable floods for river control projects and other critical points are shown in table 16. The flood volume at Chattanooga equal to 12.90 inches over the drainage area was based largely on the transposition of the storm of January 12 to 25, 1937, from the Ohio River Valley, and on the assumption that 90 percent of the rainfall would run off within the flood period. The design flood volume of 12 inches at Asheville was based on the relation between volume and peak discharge. In this case consideration also was given to the transposition of the greatest average rainfall in the storm of July 1916 over an area equal to that above Asheville, and the application to that rainfall of uniform water loss rates equal to that of the mid-August 1940 storm. Storm and flood studies indicated that amounts somewhat less than 12 inches may be used safely for areas in the upper Holston River Basin. A volume of 8.5 inches was adopted for the South Holston project, and 9 inches for the Watauga project.

TABLE 17.—*Average storm rainfall over selected areas and unregulated crest stages for the four highest floods at Chattanooga.*

Area	Average storm rainfall in inches			
	March 1-7, 1867	Feb. 23-25, 1875	March 26-April 1, 1886	January 27-February 2, 1957
Clinch River above Norris Dam	8.0	—	5.4	7.3
Holston River above Cherokee Dam	8.6	5.2	4.0	7.2
French Broad River above Dandridge	9.1	5.8	5.1	6.1
Little Tennessee River above McGhee	13.5[1]	7.6	7.7	8.3
Hiwassee River above Hiwassee Dam	12.8	6.9	7.9	7.0
Tennessee River above Knoxville	9.9	5.7	4.8	—
Tennessee River above Chattanooga	9.6	6.4	6.6	7.3
Unregulated crest stage at Chattanooga	57.9 feet	53.8 feet	52.2 feet	54 feet

1. Average above Fontana Dam.

FIGURE 76.—*Rossville Boulevard, Chattanooga, at the height of the March 1917 flood—arrow indicates height to which the 1957 flood would have risen without TVA regulation.*

CHAPTER 6

COMPUTATION OF NATURAL FLOOD HYDROGRAPHS

The preceding chapter describes methods of estimating two flood characteristics—peak rate of discharge and total flood volume. In those instances where it is necessary to know the flood volume for determining the effect of reservoir storage, it is usually also necessary to know the progressive variations in the rate of flow with respect to time. This chapter outlines the methods used by TVA in computing progressive variations in flow for natural floods and discusses flood routing. These discussions include fundamental routing procedures and—for the Tennessee River—the computation of natural storage inflow for pre-reservoir floods and discharge.

METHODS OF COMPUTING FLOOD HYDROGRAPHS

The method used to compute the progressive variations in flow will depend on a number of factors among which are the size, location, and shape of the drainage area, and whether it will be necessary to consider one reservoir or a system of reservoirs. The methods used by TVA have been developed over the years and consideration has been given to advances in the fields of hydrology and river hydraulics. Any method of computation, however, should be based on a study of past storms and floods because good agreement between computations and observed data is the best assurance that computed hypothetical flood hydrographs are reasonably correct. Two methods of computation are outlined in the following paragraphs.

First method

The first method increases daily flows of an actual flood on all parts of the drainage area and the increased flows must result in a flood volume equal to the adopted design volume. Also, after combining all components, this flood volume must equal the adopted unregulated design peak rate. The hydrograph of the Chattanooga design flood shown in figure 77 was computed by this method.

Although the tributary flow contributions to the Chattanooga design flood could have been derived from the transposition of the 1937 storm to the area above Chattanooga, it was believed that the design flood would have a sounder basis if the contributions from each sub-area were based on an actual flood. Accordingly, tributary contributions were computed by expanding the actual flood of March-April 1936.

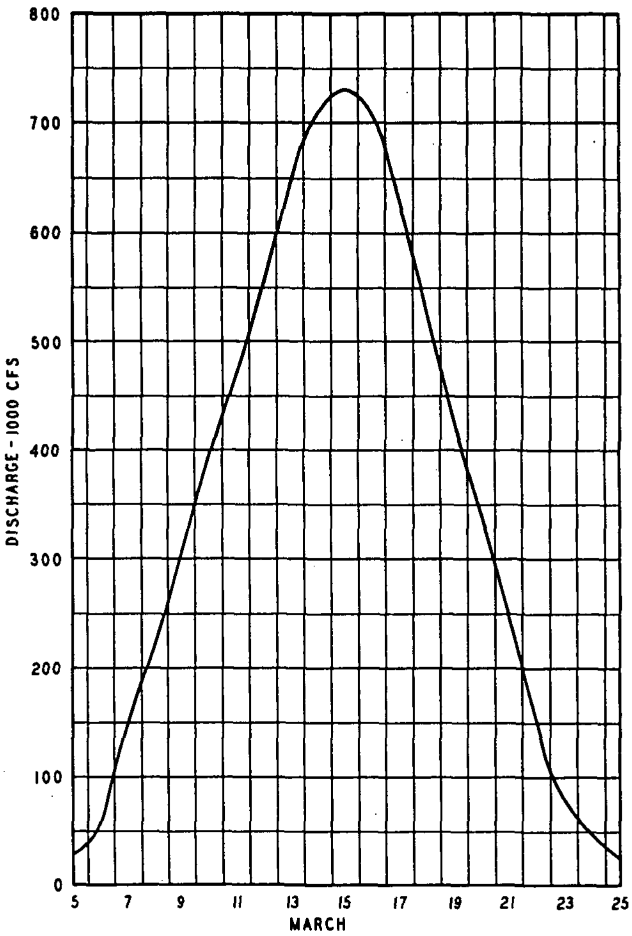


FIGURE 77.—*Maximum probable (design) flood hydrograph—Tennessee River at Chattanooga.*

TABLE 18.—*Chattanooga maximum probable flood—contribution of tributary areas.*

Date	Discharge in units of 1000 cubic feet per second: Natural discharge at tributary dam sites: Cherokee	Douglas	Fontana	Norris	Hiwassee	Other tributary contributions: To Knoxville	Knoxville to Fort Loudoun	Fort Loudoun to Watts Bar	Watts Bar to Chickamauga	Chickamauga to Chattanooga	Total inflow above Chattanooga	Cumulative inflow above Chattanooga	Natural Chattanooga discharge	Cumulative Chattanooga discharge	Date
March															March
5	13	26	11	14	4	6	4	30	6	3	117	117	29	29	5
6	34	61	17	39	5	18	23	56	6	6	265	382	58	87	6
7	48	76	21	74	7	24	21	89	21	8	389	771	147	234	7
8	69	76	27	73	9	33	29	130	32	11	489	1260	221	455	8
9	59	103	32	58	18	40	32	142	40	15	539	1799	305	760	9
10	71	104	38	71	23	42	36	150	58	21	614	2413	390	1150	10
11	78	118	69	68	61	44	38	191	81	25	773	3186	470	1620	11
12	80	158	100	110	63	42	39	204	99	29	924	4110	554	2174	12
13	93	154	75	138	41	34	18	121	127	23	824	4934	645	2819	13
14	88	100	57	82	25	23	18	66	93	19	571	5505	709	3528	14
15	63	90	50	49	25	13	11	62	81	14	458	5963	730	4258	15
16	50	83	43	38	22	8	11	52	59	11	377	6340	707	4965	16
17	47	69	32	32	18	7	9	44	49	8	315	6655	634	5599	17
18	37	54	23	29	16	5	9	22	21	4	183	7084	423	6550	19
19	28	42	20	23	13	3	7	34	34	5	246	6901	528	6127	18
20	23	34	14	20	9	3	5	13	16	3	140	7224	334	6884	20
21	11	19	7	14	7	2	5	6	12	1	84	7308	244	7128	21
22	7	11	4	9	5	1	4	4	5	1	51	7359	150	7278	22
23	4	8	3	5	4	1	4	4	2	0	35	7394	77	7355	23
24	3	4	2	2	2	1	2	4	1	0	21	7415	46	7401	24
25	1	1	1	1	1	1	1	2	1	0	10	7425	24	7425	25
Total day-second-feet (in 1000's)	907	1391	646	949	378	351	326	1426	844	207	7425	—	7425	—	
Total inches	9.84	11.40	15.28	12.12	14.52	13.53	19.68	16.19	12.50	12.61	12.90	—	12.90	—	
Drainage area, square miles	3429	4541	1571	2912	968	964	616	3277	2512	610	—	21,400	—	—	

The rainfall causing the 1936 flood occurred mostly in three separate storms with periods of little or no rain between. If these periods of little or no rainfall were eliminated, the rainfall would have been similar in duration to that of January 1937 over the lower Ohio Valley. The elimination of these periods would mean that the flows resulting from the second storm in 1936 would have occurred 4 days earlier and from the third storm, 7 days earlier. Consequently, daily tributary flows in the actual 1936 flood were graphically adjusted by the 4 days and 7 days correction and then were increased by 79 percent, so that the total flood volume would equal that determined by taking 90 percent of the transposed 1937 storm.

Depending upon the distance from the main river, a time of travel of either one-half day or one day was assumed from the tributary dam sites to the main Tennessee River where large volumes of flood storage affect the flows. The daily flows were then routed downstream through three natural reaches of the main Tennessee River—Knoxville to Fort Loudoun Dam site,[1] Fort Loudoun to Watts Bar Dam site, and Watts Bar to Chickamauga Dam site. To the routed natural flows at Chickamauga Dam site were added flows from the area below this location to the Chattanooga gage. This main river routing was accomplished with the aid of storage curves developed from topographic maps and computed natural flow profiles. An example of natural flood routing in the Watts Bar-Chickamauga reach is given under "Computation of inflow for pre-reservoir floods—Tennessee River" later in the chapter.

Flood volume contributions of each sub-area above Chattanooga in the maximum probable flood are shown in table 18, and a comparison between these and the average sub-area contributions in eleven actual floods is given in table 19. Although a relatively smaller contribution from "all other" areas was adopted for the maximum probable flood than was indicated by the average of actual floods, this is offset by the large contribution used from French Broad River where the storage reservation is small.

Second method

The second method of computing progressive variations in the rate of flow is more comprehensive than the first in that it may be applied to many small areas and by combining them, hydrographs for larger areas may be determined from rainfall. The method of computation consists of (1) an analysis of rainfall, rainfall losses, and stream flow in several actual storms and floods, and the determination of a relation between rainfall, less losses, and discharge; and (2) the application of that relation to hypothetical rainfall or runoff of sufficient quantity to produce the maximum probable flood. This method was used to compute flood hydrographs in the French Broad River area above Asheville and in several other areas. These hydrographs are shown in figure 78.

1. The term "site" is used to distinguish natural discharge from discharge at the dam.

TABLE 19.—*Comparison of tributary contributions in Chattanooga maximum probable flood with average contributions in actual floods.*

Tributary	Drainage area, percent of drainage area above Chattanooga	Tributary contribution in maximum probable flood, percent of Chattanooga volume	Average tributary contribution in actual floods, percent of Chattanooga volume
Cherokee	16.0	12.2	11.9
Douglas	21.2	18.7	14.1
Fontana	7.3	8.7	8.0
Norris	13.6	12.8	14.8
Hiwassee	4.5	5.1	4.4
All other	37.4	42.5	46.8
Total	100.0	100.0	100.0

NATURAL FLOOD ROUTING

Many methods for routing floods have been developed. In fact, almost every engineer who has studied the subject has derived for his specific need either a complete procedure or a new step which may be an improvement of some older method. Detailed procedures, both analytical and graphical, are given in many text books, hydraulic hand books, and publications of the American Society of Civil Engineers and the American Geophysical Union. Some of these are of general application, while others have a special application or give only partial results, such as discharge but not storage or water surface elevation. Machines have also been devised by which the discharge hydrograph is produced if the storage-discharge relation and inflow are set up in the machine. A slide rule may also be made from which the discharge may be computed periodically when inflow and the storage-discharge relation are known.

Most methods are based on the simple fundamental relation known as the storage equation and which, stated briefly, is: *Inflow minus outflow over a given period of time is equal to the change in storage.* The routing procedure used for nearly all TVA studies of past floods and for computations of reservoir effects is based on the above relation. This procedure was suggested from the article in *Civil Engineering,* February 1931, "Rapid Calculation of Reservoir Discharge," by R. D. Goodrich. This method, in which the relation between reservoir storage and discharge is a single line readily computed from the spillway discharge and reservoir storage, was originally used for free overflow spillway reservoir routing. It has since been adapted in TVA for use with sloped profile storage applying both to natural main-river reaches and to main-river reservoirs. When so adapted a family of routing curves is used.

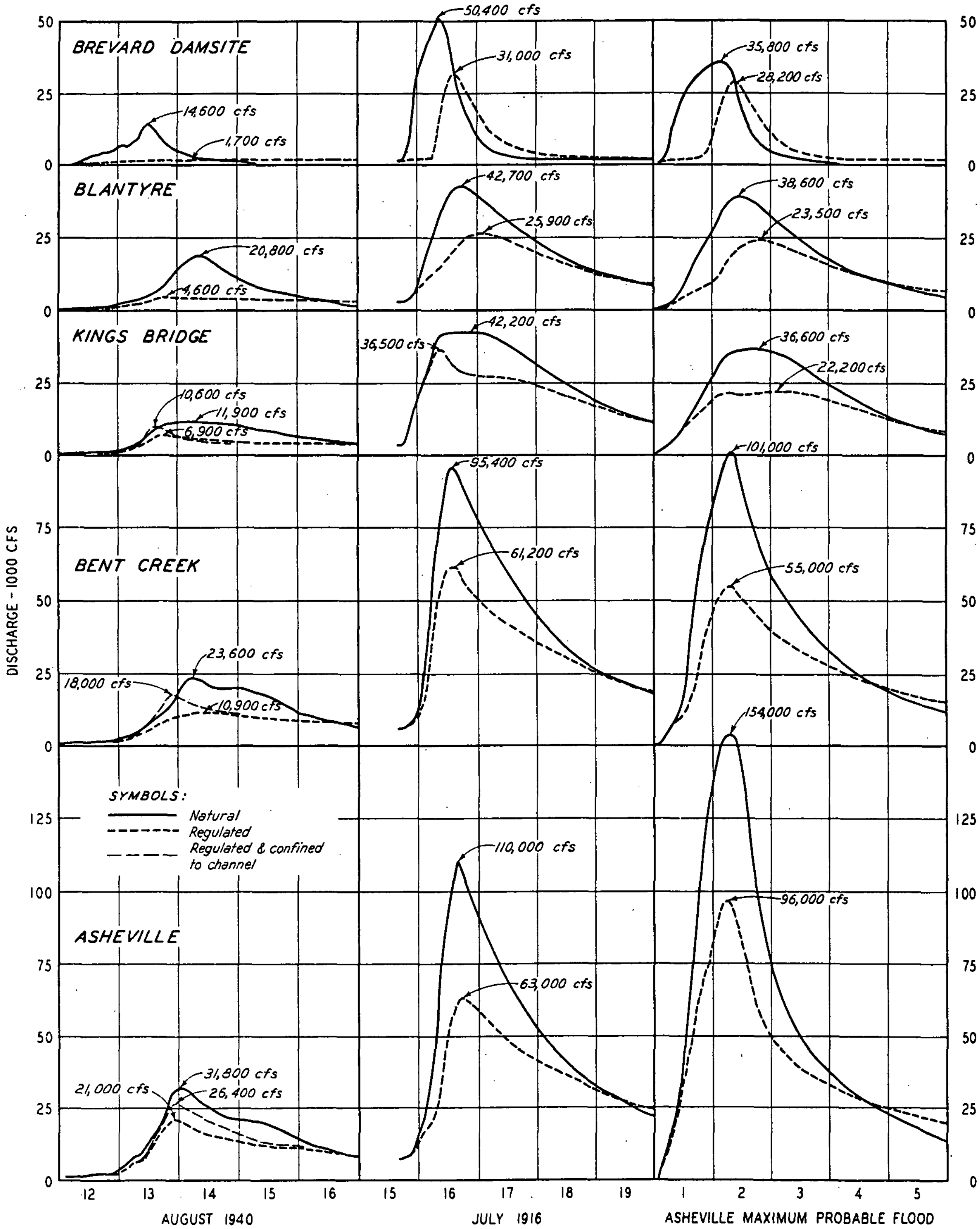


FIGURE 78.—*Natural and regulated flood hydrographs—upper French Broad River.*

Fundamental procedures

The storage equation may be stated as follows:

$$\left(\frac{I_1 + I_2}{2}\right) t - \left(\frac{O_1 + O_2}{2}\right) t = S_2 - S_1$$

in which

I is the inflow rate
O is the outflow rate
S is the volume in storage
t is the *length* of the time *interval*

and

subscripts 1 and 2 refer to the beginning and end of the time interval, respectively.

If the unit of flow is cubic feet per second and the time period, t, is measured in days, the terms

$$\left(\frac{I_1 + I_2}{2}\right) t \text{ and } \left(\frac{O_1 + O_2}{2}\right) t$$

give the total inflow and outflow in day-second-feet. The time interval must be selected so that the average flow for the interval is represented by the mean of the discharge rates at the beginning and end of the interval.

Conversion of day-second-feet to acre-feet, a common unit of storage, is accomplished by the application of the multiplying factor 1.983, or practically 2.

Multiplying the inflow and outflow terms by 2, the equation becomes

$$t(I_1 + I_2 - O_1 - O_2) = S_2 - S_1$$

with

S in acre-feet.

Dividing by t and transposing, the equation may be written

$$I_1 + I_2 + \frac{S_1}{t} - O_1 = O_2 + \frac{S_2}{t}$$

When the routing involves streams of large area the time period of one day is appropriate. On smaller streams a shorter time period should be used, and the above equation must be modified accordingly. For example, for a time period, t, of 2 hours or 1/12 of a day, the equation becomes

$$I_1 + I_2 + 12S_1 - O_1 = O_2 + 12S_2$$

still with

inflow, I, and outflow, O, in units of cubic feet per second

and

storage, S, in units of acre-feet.

The first problem is to determine a relation between O + S, O, and other variables. There are two principal methods for computing the amount of storage in a reach or basin, and the method used depends on the size of the stream being studied and the data available.

In one method storage may be determined by comparing actual observed discharge with inflow computed from rainfall excess. The summation of successive differences between inflow and outflow is the storage, and this usually can be plotted against the corresponding successive inflows and outflows. This method is used when routing in small and medium-sized basins, say less than 5,000 square miles. Volumes thus determined represent storage in all stream channels, both large and small, in fact, any storage that retards the flow is included automatically. The computation of the inflow used to determine storage depends on several assumptions which are given in greater detail in the example later in this chapter.

In the other method, storage is determined from topographic maps and computed flow profiles. The reach of the river on which the routing is to be done is divided into successive incremental sections and then, after the contours have been planimetered, elevation-volume curves are constructed for each section. Next, profiles for a series of equal inflows and outflows are drawn for the reach, and then storage under each flow profile is determined by taking the sum of all the increment volumes. A routing curve (fig. 79) is then constructed usually with the discharge, O, plotted against discharge plus storage, O + S, and with flow at the upper end of the reach as the third variable. Storage obtained by this method is that in the main-river channel and flood plain only and does not include storage on the smaller tributaries nor on the ground surface. Inflow into the reach, therefore, must conform to this condition.

After the inflow has been computed and the storage (or routing) curves have been constructed, the routing procedure is the same in each of the two cases described.

Natural routing on Tennessee River

Computation of natural flood hydrographs for the Tennessee River was necessary (1) to verify computed inflows for pre-reservoir floods which could be used later for studies of reservoir operations of those floods, (2) to determine natural discharges of hypothetical design floods, and (3) to determine natural unregulated discharges of post-reservoir floods.

Adopted routing reaches in the Tennessee River were chosen to coincide with the reservoirs so that a direct comparison could be made between natural and regulated discharges.

In any reach it is necessary to know (1) flow at the upper end of that reach, (2) the contribution from any large tributaries and from all remaining areas tributary to the reach, and (3) the relation between inflow, outflow at the lower end of the reach, and storage in the reach.

Computation of natural storage—Tennessee River

Storage under flow profiles representing equal inflow and outflow was obtained from the summation of increment volume curves (usually for two miles of river) using the profile elevation at the mid-point of the increment to determine the volume in that short reach. The equal inflow-outflow profiles were based on computed natural flow profiles, and the two-mile

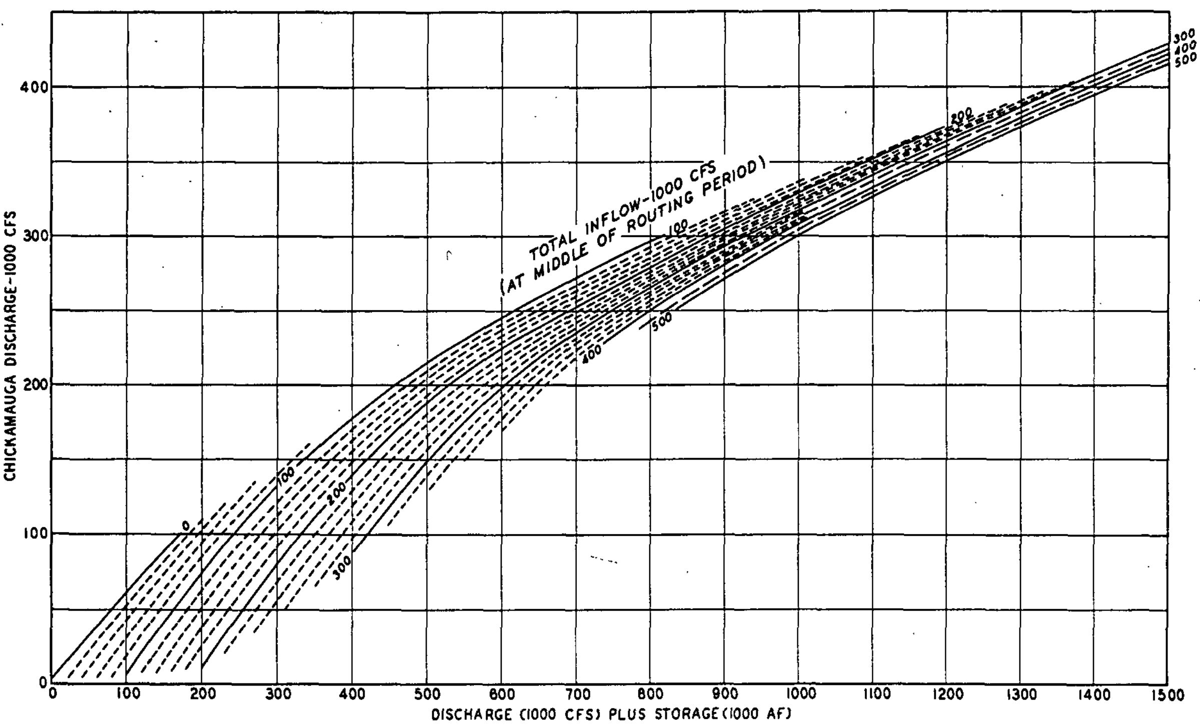


FIGURE 79.—*Relation of inflow, outflow, and storage—natural conditions—Watts Bar to Chickamauga (these curves are used in computation of discharge—Chickamauga Reach—table 21).*

TABLE 20.—*Computation of local inflow in Chickamauga reach.*

Date		Average rainfall, inches per day	Loss, inches per day	Runoff,[1] inches per day	Runoff,[1] cubic feet per second	Runoff, in cfs, occurring on successive days according to following percentages						Base flow, cfs	Total local flow, cfs
						27	37	19	10	5	2		
February	28	0.41	0.22	0.19	6,050	1,630	—	—	—	—	—	4,740	6,370
March	1	1.39	.76	.63	20,000	5,400	2,240	—	—	—	—	4,740	12,380
	2	1.19	.65	.54	17,200	4,640	7,400	1,150	—	—	—	4,740	17,930
	3	1.30	.71	.59	18,800	5,080	6,360	3,800	610	—	—	4,740	20,590
	4	1.24	.67	.57	18,200	4,910	6,960	3,260	2,000	300	—	4,740	22,170
	5	.11	.06	.05	1,590	430	6,730	3,570	1,720	1,000	120	4,740	18,310
	6	—	—	—	—	—	590	3,460	1,880	860	400	4,740	11,930
	7	.12	.06	.06	1,910	520	—	300	1,820	940	340	4,740	8,660
	8	—	—	—	—	—	710	—	160	910	370	4,740	6,890
	9	—	—	—	—	—	—	360	—	80	360	4,740	5,540
	10	—	—	—	—	—	—	—	190	—	30	4,740	4,960
	11	.21	.11	.10	3,180	860	—	—	—	90	—	4,740	5,690
	12	.24	.13	.11	3,500	940	1,180	—	—	—	40	4,740	6,900
	13	.19	.10	.09	2,860	770	1,290	600	—	—	—	4,740	7,400
													155,720

1. Using a flood runoff of 45.7 percent of rainfall and drainage area of 1,182 square miles.

increment volume curves were computed by planimetering topographic maps.

Next, to obtain storage for unequal inflow and outflow profiles, it was assumed that all of an inflow increase would add to the storage in a reach except that part which, during a routing period, would flow on through and out the lower end, thus increasing the discharge. The relative amounts of increased storage and increased outflow depend on the length of reach and the time of travel. In a reach in which the time of travel was greater than the routing time interval, nearly all the inflow increase would be added to the storage. The amount going into storage as a result of the increased inflow was added to the known storage for the equal inflow-outflow conditions to obtain a storage value for the unequal inflow-outflow conditions. A sufficient number of variations can be assumed to include all conditions of rising inflow likely to be encountered. Similarly, a series of decreasing flows may be assumed to provide storage values suited to anticipated needs.

Figure 79 gives the relation between inflow, outflow, and outflow plus storage for Chickamauga reach, and an example of its application in this reach follows.

Computation of inflow for pre-reservoir floods—Tennessee River

Discharge at the upper end of the reach is assumed to be known either from observed stages at a gaging station or from the results of previous upstream routing computations. Contributions from the area between the two ends of the reach are obtained from gage records of the flood to the extent they are available, and the remainder are computed from rainfall.

The procedure for computing the inflow from rainfall assumes (1) that the total volume of runoff can be determined from gaging station records, (2) that sufficient rainfall stations are available to determine average daily rainfall, and (3) that the distribution of runoff with respect to time is known. Inflow for hypothetical storms for which no gage records are available may be computed entirely from rainfall excess and the assumed distribution.

The following computations are examples of a local inflow determination and natural flood routing in the Chickamauga reach of the Tennessee River. In the period of 14 days in the example, gages on the Tennessee River and on the Clinch, Little Tennessee, and Hiwassee Rivers indicated that the runoff from the local area of the Watts Bar-Chickamauga reach was about 150,000 day-second-feet. A base flow of 4 cubic feet per second per square mile was also indicated by initial flows on the tributaries and main river. Subtracting the base flow for the 14-day period, equal to 66,000 day-second-feet, from the total flow, there remains 84,000 day-second-feet which came from rainfall. This is equivalent to a flood runoff of 45.7 percent. A computation of local inflow is shown in table 20, using rainfall distributions of 27 percent on the day of occurrence of rain, 37 percent on the second day, and 19, 10, 5, and 2 percent on the following days.

Computation of discharge—Tennessee River

With all the necessary preliminary computations made, the inflow may be routed with the aid of the storage plus discharge curves (fig. 79) to determine the discharge. This computation may be made as shown in table 21, and further explained in the following paragraph.

All inflows are considered as rates of flow at the time indicated and are recorded in their appropriate positions in the tabulation. The sum of the inflows, I_1, is the inflow rate at the beginning of the routing period. The inflow at the end of the period, I_2, is of course the same as the inflow at the beginning of the next period and is entered under I_1 of that next period. The outflow, O_1, for the first period, February 28, is assumed or made equal to the observed flow for that date. If there had been no prior flood the initial outflow could be made equal to the initial inflow, but in the example the initial outflow was 82,000 cubic feet per second. Next, to determine the initial storage, S_1, the family of curves in figure 79 is entered with the initial outflow and inflow of 82,000 cubic feet per second for the middle[1] of the preceding period to read a value for outflow plus storage of 199,000. Subtracting the initial outflow, O_1, from this value gives the initial storage, S_1, as 117,000 acre-feet.

From the sum of I_1, I_2, and S_1 is subtracted the outflow, O_1, to obtain the relation $O_2 + S_2$, equal to 191,000. The family of curves is then entered with this value and the inflow at the middle [1] of the period to determine the outflow at the end of the period of 79,000 cubic feet per second. Subtracting this outflow from the value $O_2 + S_2$, gives a storage for the end of the period of 112,000 acre-feet. Subsequent operations are similar and the process is repeated, no further assumption being necessary.

Except in special cases, routing on the main river usually commences at Knoxville. Tributary flow into the Knoxville-Fort Loudoun reach is added to discharge at Knoxville and the sum is routed to determine outflow at Fort Loudoun Dam site. Appropriate tributary flow is added to this outflow, becoming the inflow into the next downstream reach. This process is continued to the mouth of the river. When studying a known flood, computed hydrographs are compared with observed hydrographs at gaging stations and, if appreciable differences exist, local inflows are then adjusted to the extent necessary. After agreement between actual and routed hydrographs has been attained, then the effect of modifications in flows caused by operation of tributary or main-river storage reservoirs may be carried downstream.

1. Inflow at the middle of the period has been used for entering the routing curves because it helps to even out the effect of extreme changes in inflow.

TABLE 21.—*Computation of discharge—Chickamauga reach.*

	Unit	Period													
		Feb.	March												
		28	1	2	3	4	5	6	7	8	9	10	11	12	13
Watts Bar discharge	1000 cfs	66	59	99	155	216	262	304	299	238	168	117	89	67	77
Hiwassee River at Charleston	1000 cfs	6	7	21	23	38	54	47	36	30	21	12	11	10	8
Local inflow	1000 cfs	6	12	18	21	22	18	12	9	7	5	5	6	7	7
Total inflow, I_1	1000 cfs	78	78	138	199	276	334	363	344	275	194	134	106	84	92
Total inflow, I_2	1000 cfs	78	138	199	276	334	363	344	275	194	134	106	84	92	—
Storage, S_1	1000 ac-ft	117[1]	112[6]	150	243	367	515	655	734	704	564	372	219	144	125
$I_1 + I_2 + S_1$	1000's	273[2]	328[7]	487	718	977	1212	1362	1353	1173	892	612	409	320	—
Outflow, O_1	1000 cfs	82[3]	79[5]	99[9]	145	206	256	301	327	322	287	233	160	105	90
$O_2 + S_2$	1000's	191[4]	249[8]	388	573	771	956	1061	1026	851	605	379	249	215	—

Add components making up total reach inflow.
Follow steps (curves mentioned are shown in figure 79):
1. Starting storage—from volume curves for initial conditions.
2. Add known quantities, $I_1 + I_2 + S_1$ for February 28.
3. Starting outflow—assumed.
4. Subtract assumed initial outflow to get $O_2 + S_2$ for February 28.
5. Enter curve with inflow and $O_2 + S_2$ to find O_2 for February 28—this is entered as O_1 of succeeding day (March 1).
6. Subtract O_2 from $O_2 + S_2$ to get S_2 for February 28, but enter as S_1 of succeeding day (March 1).
7. Add known quantities for March 1 ($I_1 + I_2 + S_1$).
8. Subtract known O_1 to get $O_2 + S_2$, both for March 1.
9. Enter curve with inflow and $O_2 + S_2$ to find O_2 for March 1—this is entered as O_1 of succeeding day (March 2).

Continue by repeating day-by-day these procedures for the entire flood.
Note: In reading the routing curves (fig. 79), average of two days inflow is used to help smooth out rapid changes in inflow.

CHAPTER 7

STUDIES AND PRINCIPLES OF FLOOD CONTROL OPERATIONS

In planning the method of operating the reservoir system a number of principles and objectives emerged, and this chapter 7 discusses the many factors involved in their determination and attainment. Chapter 8 then describes the present system, chapter 9 discusses its actual operation, and appendix B gives results of its actual operation during floods.

Following a brief summary of the flood situation in the Valley and on the lower Ohio and Mississippi Rivers before TVA, the chapter itemizes the reservoir operating principles and objectives which emerged as planning progressed and operating experience developed, and discusses the principal factors considered and studied in their determination. These factors include the effect of section 7 of the Flood Control Act of December 22, 1944. The remainder of the chapter discusses fixed-rule and ideal reservoir operation, and the coordination of flood control with operations for navigation, power, malaria control, and recreation.

The TVA Act provides for the control of destructive flood waters both in the Tennessee River Basin and in the Mississippi River Basin.

In general, all the valleys in the Tennessee River Basin were subject to periodic flooding before the TVA reservoir system was built. Over most of the region this flooding involved mainly the inundation of farm land with resulting damage to crops at certain times of the year. Nevertheless, there were numerous points of danger in the Tennessee River Basin where encroachment on the flood plain by cities and towns had created serious hazards and increased greatly the amount of damage suffered from floods. Chattanooga presented the most serious flood situation. To the end that this situation might be remedied, or at least greatly alleviated, the studies of reservoirs above that city were primarily directed.

The Tennessee River was often a major contributor to Ohio and Mississippi River floods—particularly before the completion of Kentucky Dam near the mouth of the river. Consequently, studies for a reservoir at that location were primarily concerned with reservoir regulation that would reduce the crests of these floods on the lower Ohio and Mississippi Rivers.

OPERATING PRINCIPLES AND OBJECTIVES

The principles and objectives which evolved as operating experience was gained during the progress of planning and constructing the early projects in the system were necessarily tentative and not developed in detail. It is noteworthy, however, how accurate they were and how well they served in formulating the present methods of operation of the larger system.

The use of the multiple-purpose type of project in the Tennessee River Basin is implied in the TVA Act by the provision for the construction of dams and reservoirs to afford a 9-foot navigation channel in the Tennessee River and to control destructive floodwaters in the Tennessee and Mississippi River drainage basins. In addition, the Act authorizes the operation of facilities for the generation, transmission, and sale of power that would be created at the dams.

As plans for the much needed reservoirs progressed and as experience was gained in the actual operations of the early projects, the following operating principles and objectives emerged:

1. The use of reservoirs between certain elevations for dual purposes is feasible in the Tennessee Valley because the annual critical flood season at Chattanooga is quite definite as to time of year, from about December 15 to April 15 of the following year.
2. During periods of substantial flood flows, the operation of the reservoir system above Chattanooga will be primarily for reduction of flood stages at Chattanooga and other points in the upper Tennessee Valley, with incidental downstream benefits.
3. The flood control operation of the Tennessee River reservoirs below Chattanooga is primarily for the reduction of flood heights along the upper reaches of those reservoirs and to supplement Kentucky Reservoir operations.
4. The flood control operation of Kentucky Reservoir is primarily for the reduction of flood

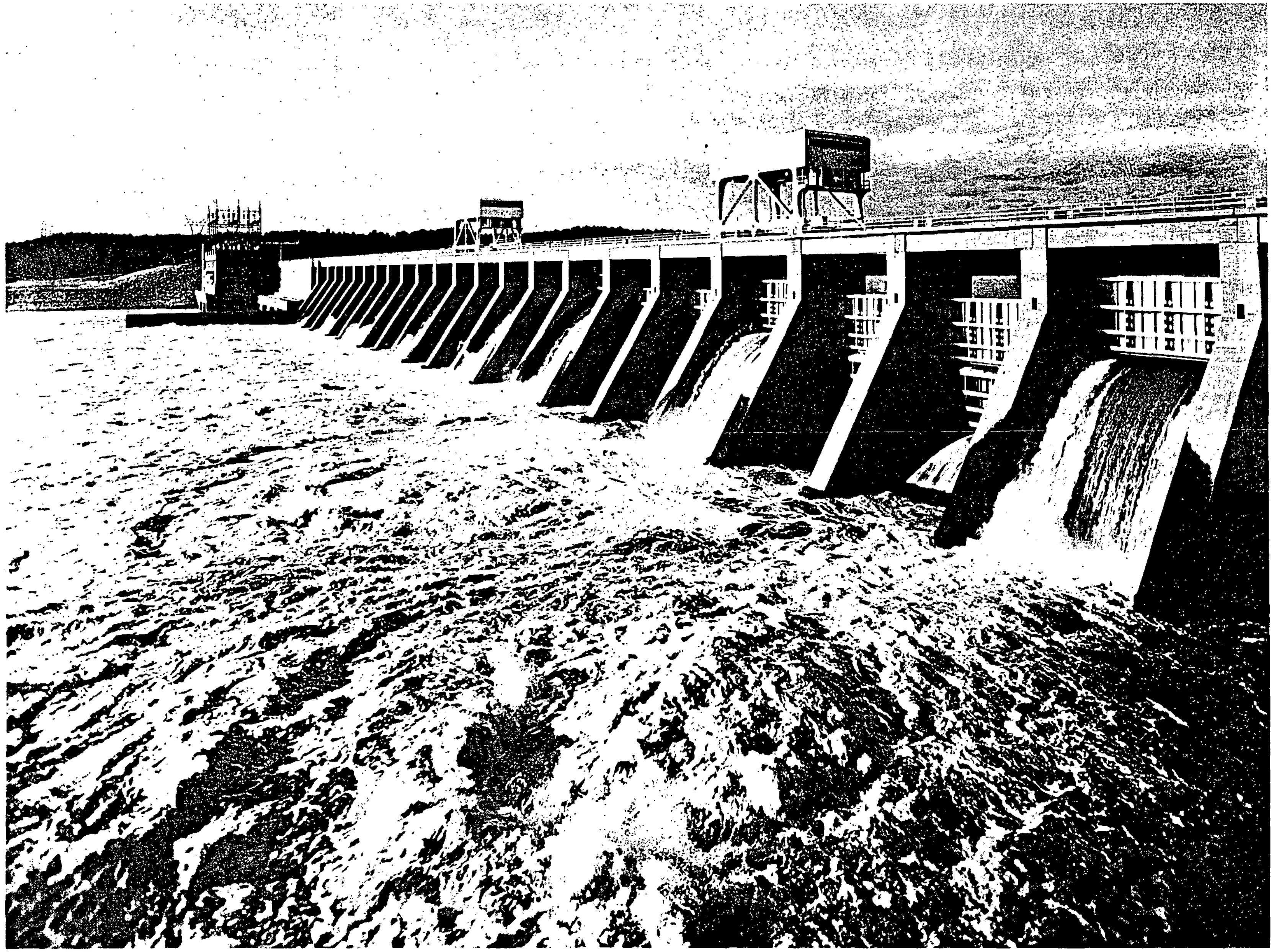


FIGURE 80.—*Pickwick Landing Dam, a main river project, during a flood control operation.*

heights along the lower Ohio and Mississippi Rivers.

5. An available capacity of at least 4,000,000 acre-feet, suitably distributed among the tributary reservoirs above Chattanooga, is to be reserved for flood control during the flood season through March 15, with a gradually decreasing reservation permissible between March 15 and April 15. Thus, flood storage capacity to April 15 will be operated primarily for protection of Chattanooga. After this date, it will be largely for the protection of agricultural lands from the smaller, crop-season floods. Additional flood storage capacity will be available on January 1 to allow for multiple floods, with the amount decreasing from January 1 to 4,000,000 acre-feet on March 15.
6. Emptying the reservoirs to the normal filling level is to be timed so as not to interfere with the flood control operation of downstream projects, particularly at Kentucky Reservoir.
7. The total available capacity of the tributary reservoirs now built is not sufficient for the complete storage of the inflow in the largest floods which may be expected, and as the present technique of weather forecasting cannot furnish sufficiently accurate indication on the length or depth of rainfall and the consequent magnitude of a flood, some release from the tributary reservoirs during large floods will be advisable or necessary.
8. In the mainstream reservoirs, except in Kentucky, a storage reservation of from 2 to 10 feet is provided for flood control throughout the flood season. In Kentucky the reservation is 21 feet. This reserved space is sufficient for the retention of only a limited portion of the flow in the Tennessee River and, therefore, the available storage capacity, as far as possible, must be held for use when the peak on the Ohio arrives.
9. The objective in the use of the reservoir system for flood control is for the regulation of damaging floods only, and the available capacity is to be reserved for that purpose.
10. Data on rainfall and streamflow will be collected continuously during the progress of all storms and utilized as a guide to daily detailed operation. Successful reservoir control will require not only adherence to the general principles, but also close attention to and prompt allowance for the daily developments.

The concept of operating the multiple-purpose reservoirs for flood control also visualizes the use of the tributary reservoirs to control floods in the tributaries, leaving the mainstream reservoirs to regulate the flood inflow in the main-river drainage area plus the reduced flow from tributary reservoirs. By withholding water in the tributary reservoirs until after the regulated peak from the main-river drainage area has passed Chattanooga, the burden on the mainstream reservoirs, which have much less flood storage space, is lightened.

The successful regulation of a flood requires careful coordination of the operation of the tributary and mainstream reservoirs. It also requires that the prescribed flood storage space be held in reserve during the entire flood season except when used for regulating floods. Thus, operations in the late summer and fall drawdown period, as well as during the actual flood season, have an important influence on subsequent flood control. All water stored in the reservoirs above the flood season levels in the fall must be withdrawn before the end of the year. Operation in the months preceding the flood season is as essential in the subsequent reduction of floods as the proper operation of reservoirs during a flood.

All reservoirs on the main stem of the Tennessee River—except Hales Bar—are used to supplement the tributary reservoirs in regulating floods. At Hales Bar the available volume of storage space is too small to be of material value for this purpose. Advantage is taken of the flexibility of this small pool, however, by drawing it down to reduce backwater effect at Chattanooga in small floods. In larger floods this benefit is lost because the backwater control for high flows is in the river gorge section above the dam.

The determination of the operating principles and objectives outlined and discussed briefly in the preceding paragraphs required consideration and study of many factors that would affect or influence flood control operations. The principal of these factors are discussed in the remainder of this section.

Seasonal occurrence of great Valley-wide floods

An important and controlling element in planning the dams and reservoirs was the seasonal character of major Valley-wide floods in the Tennessee Valley. The record—now some 90 years long, though not entirely complete, and extending back to a total of about 150 years with historical high water marks—indicates that the large, destructive, Valley-wide floods occur during a more or less well-defined period of about four months. This is due to high and generally concentrated periods of precipitation and to the high runoff-rainfall ratio in the winter and early spring when vegetation and foliage are dormant. This flood period extends from about the last half of December to the first part of April in the following year, as indicated by the seasonal plot of floods in figure 25, page 22, "Distribution of floods, Tennessee River at Chattanooga, Tennessee." Between the middle of April and the middle of December the floods which may occur are either not large or are confined to relatively small areas. This condition is due to the limited geographic extent of intense precipitation during this season, as well as the comparatively low runoff from heavy rains caused by the interception, retention, and transpiration of water by growing vegetation and foliage, the low moisture content in the soils, and high evaporation rates.

The seasonal flood characteristics in the Tennessee Valley, as explained above, led to the early adoption of an annual cyclical operation plan. Under this plan all reservoirs are drawn to a low level by the beginning of the flood season near the first of January, thus providing storage space for flood control. As the ensuing flood season advances, the reservoirs on the tributaries are allowed to fill gradually until about the first of April. After flood crests have subsided, surplus stored floodwater is released as necessary until storage space is again available should a subsequent flood occur. Figure 113 on page 179 is a chart showing the actual operation of Douglas Reservoir in 1946 when a February flood followed one in January. After the end of the flood season, the tributary reservoirs are filled to the highest levels of the year about as rapidly as water is available. Then for a period of a few months, depending on the availability of runoff, these reservoirs are maintained at their relatively high level preparatory to drawdown to minimum flood season level by the end of the year. If feasible, all the water withdrawn during the summer and fall is used in the production of power, supplementing low flows normally occurring in these seasons.

Floods in small areas are not confined to a particular season as is the case with great Valley-wide Tennessee River floods. The greatest floods on some of the tributaries have occurred in almost any season of the year. For example, at Elizabethton, Tennessee, the highest known flood occurred in May, the second highest in August, and the third and fourth highest in February and March. The French Broad River Basin above Asheville, North Carolina, is subject to intense hurricane rainstorms which occur in the summer and fall. The greatest flood at Asheville occurred in July 1916 as a result of such a storm. The planning of flood protection in these small areas must provide for the fact that floods occur in any month.

Relation of tributary reservoirs to Chattanooga

Approximately half of the area of the Tennessee River Basin is located above Chattanooga in the hilly and mountainous regions in eastern Tennessee, southwestern Virginia, western North Carolina, and northern Georgia, where the valleys and the river channels are relatively narrow. This part of the Basin is drained by the Tennessee River's five largest tributaries, the Hiwassee, Clinch, Little Tennessee, French Broad, and Holston River, and by the upper 180 miles of the main stem itself. The areas drained by these streams are shown in table 22. Each of these areas, separately or jointly with one or more of the other areas, has contributed heavily to past Chattanooga flood crests. Therefore, it was desirable that flood runoff from all the areas and as much of each of them as practicable be controlled by reservoirs. At least one reservoir with flood storage reservation was built on each of the five tributary streams. The tributary reservoirs thus provided, together with the drainage areas lying upstream, are listed in table 23.

TABLE 22.—*Drainage areas of tributary streams and main stem above Chattanooga.*

River	Drainage area, square miles
Hiwassee	2,700
Clinch	4,413
Little Tennessee	2,627
Holston	3,776
French Broad	5,124
Total of tributaries above Chattanooga	18,640
Main stem above Chattanooga	2,760
Total above Chattanooga	21,400

The flood runoff from some of these tributary drainage areas is more effectively controlled than from others. For instance, the flood storage capacity provided by Douglas Reservoir during the flood season is equivalent to only about 4 to 5 inches of depth on the drainage area, but about twice that depth is provided by Norris Reservoir. Moreover, the total area controlled by all the tributary reservoirs is only about 72 percent of the total tributary drainage area and about 63 percent of the total drainage area above Chattanooga. This leaves the runoff from about 8,000 square miles, plus any water released from the tributary reservoirs, to be regulated by mainstream reservoirs.

A large portion of the city of Chattanooga, including much of the business section, is located on low ground partially surrounded by hills and mountains. Generally speaking, the riverbanks are about 30 feet high. Above and beyond the banks lies the built-up city in which damage from inundation mounts rapidly as water stages increase. In the past, before TVA started its flood control operations with the completion of Norris Dam in 1936, heavy rain-

TABLE 23.—*Drainage areas controlled by tributary flood control reservoirs.*

River	Reservoirs	Drainage area, square miles
Hiwassee	Hiwassee Chatuge Nottely	968
Clinch	Norris	2,912
Little Tennessee	Fontana	1,571
Holston	Cherokee Boone Watauga South Holston	3,428
French Broad	Douglas	4,541
Total tributary area controlled above Chattanooga		13,420
Percent of total area above Chattanooga		63

fall on the drainage area of 21,400 square miles above the city caused damaging floods of varying degree in the city. The highest flood occurred in March 1867 when the river rose to a stage of 57.9 feet, or about 28 feet above the top of bank. Other great floods occurred at this point in 1875, 1886, and 1917, when the crest was some 17 to 24 feet higher than the top of bank. These and other damaging floods which occurred, together with the increasing damages which flooding causes in the rapidly growing city, focused attention on this city as the greatest flood hazard in the Valley. Chattanooga thus became a key point in planning the reservoir operations.

Critical Cairo floods

The highest flood of record at Cairo occurred February 3 and 4, 1937, when the stage reached 59.5 feet (about elevation 330) on the gage, about a week after the Birds Point-New Madrid floodway [1] was opened near stage 58 feet to alleviate the Cairo flood situation. Figure 81 shows scenes in the floodway following subsidence of the flood and figure 82 shows Cairo during the flood. This same flood produced the maximum stage of record on the Ohio River at Paducah. Cairo was seriously threatened. Mud boxes placed on top of levees were nearly overtopped. Since that flood, the levees have been strengthened and raised to a grade of 64.6 feet. With the use of the Birds Point-New Madrid floodway they will safely carry the project flood of 2,450,000 cubic feet per second, about 20 percent greater than the 1937 flood.

Damaging stages

Discharge capacity of the stream channel at points where damage from floods begins is usually small in relation to the peak discharge of the maximum probable flood, or even to that of the maximum known flood. The reduction of a flood to a non-damaging stage is the ultimate goal of flood control by storage in reservoirs. If levees can be provided economically, the non-damaging stage is raised and a greater discharge can be passed without damage than in the natural channel, and as a result less flood storage in reservoirs is required.

A profile of the damaging stages at various points is difficult to make because they may vary widely even within short distances along the stream. Moreover, the type of property under consideration may be affected at a higher or lower stage than other types of property. For example, at Chattanooga land lying along Chattanooga Creek (some of which is used for agriculture) begins to be flooded at a stage of 20 feet. The "flood stage" adopted by the U. S. Weather Bureau some years ago is 30 feet, and a few small houses are flooded at this stage. Significant industrial, commercial, and residential damage does not begin, however, until a stage between 32 and 33 feet is reached. Likewise, at Savannah, Tennessee, the flooding of farm land begins at about elevation 368, or a stage of 16.7 feet. The published U. S. Weather Bureau flood stage, however, is 39 feet, some 22 feet higher. Similar differences appear at many other points in the Basin, and it is not practicable, therefore, to present a comprehensive list or chart of damaging flood stages.

Operation in possible large summer flood

As previously described, the multiple-purpose reservoirs on the tributaries and on the main stream are filled, or partially so, after the end of the winter flood season, so that there is considerably less storage space available in these reservoirs during the summer months than during the winter months. The volume of runoff from a summer storm is relatively less than from a winter storm of the same magnitude because of the greater water losses due to evaporation, infiltration into the soil, and plant consumption. Nevertheless, substantial storage space is still reserved, particularly in Norris, South Holston, and Watauga Reservoirs on tributaries, and the mainstream Kentucky Reservoir, for use in the regulation of a summer flood.

Table 26 in chapter 8 shows the minimum amount of storage space available in the various reservoirs during the summer months. In the case of the mainstream reservoirs, it is sometimes feasible in regulating summer floods to draw the reservoir level down temporarily in advance of the arrival of the flood peak, thus providing additional space for use in its reduction.

It was evident in planning the operation of Kentucky Reservoir that there would be little if any occasion to use the reservoir above elevation 365 for storage of floods during the season from June 1 to December 1, as floods during this time are infrequent and relatively small. This would permit farming operations to continue in the zone between elevation 365 and 375. Accordingly, it was decided that where flowage easements could reasonably be applied in this zone, owners would be left in possession of the fee title to the land, and the right to flood by the reservoir would be limited to the period December 1 to June 1 of the following year, which embraces the major flood season on the Tennessee, Ohio, and Mississippi. This procedure resulted in minor savings in the cost of reservoir lands, but permitted a large area of land to remain in production.

Effect of Section 7, Flood Control Act of December 22, 1944

Section 7 of the Flood Control Act of December 22, 1944, provides that ". . . in case of danger from floods on the Lower Ohio and Mississippi Rivers the Tennessee Valley Authority is directed to regulate the release of water from the Tennessee Reservoir into the Ohio River in accordance with such instructions as may be issued by the War Department." This

1. H. Doc. No. 359, 77th Cong., 1st sess., pp. 13-14: "*Northern section—Cape Girardeau to White-Arkansas*— . . . Flowage (rights) required over lands in the Birds Point-New Madrid floodway has been acquired, except for a few tracts, condemnation suits for which are now in the courts."

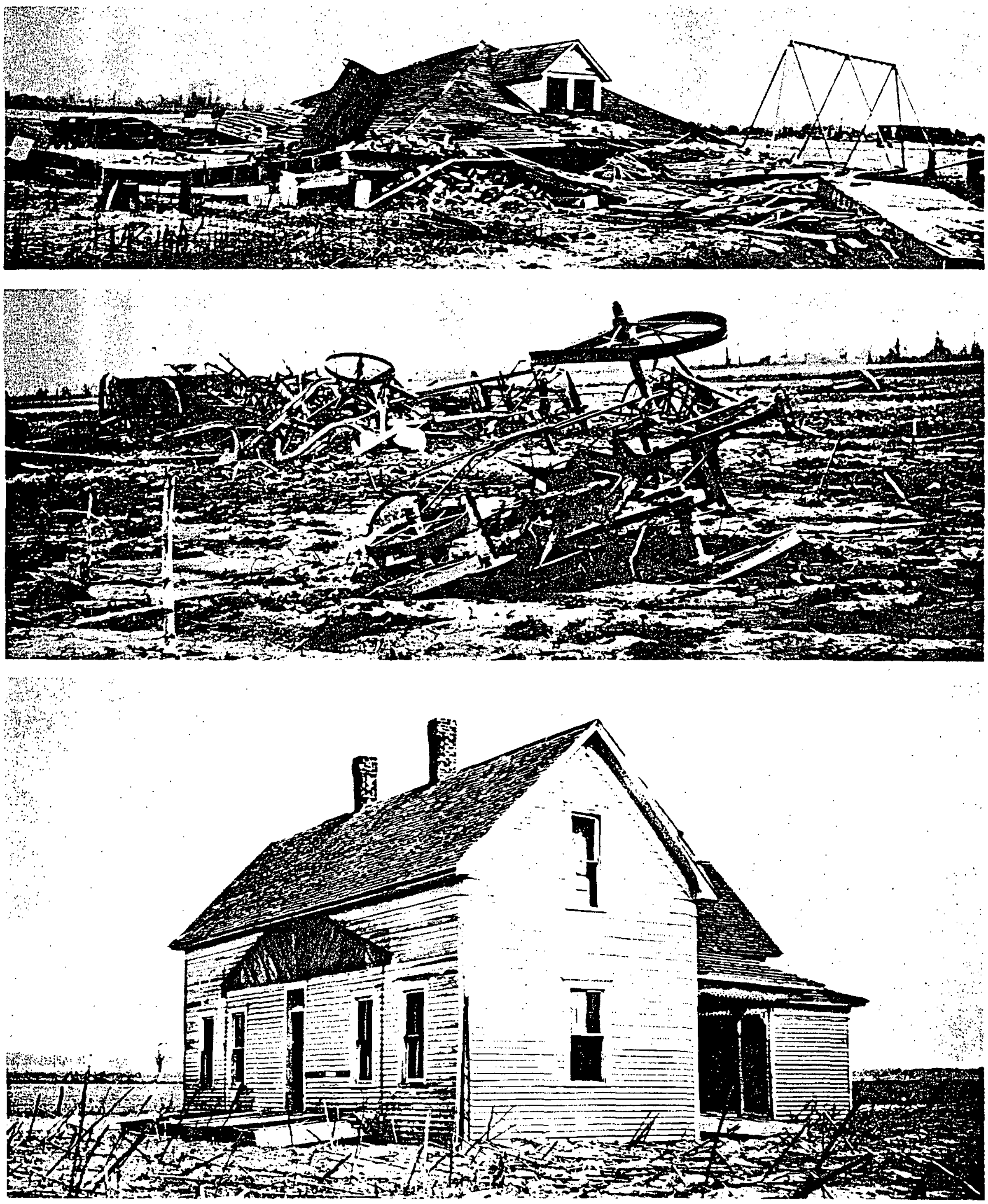


FIGURE 81.—*Birds Point-New Madrid floodway after it had been opened and used in 1937 to alleviate flooding at Cairo. Top view: Wrecked school building. Middle view: Farm machinery battered and mangled out of shape. Bottom view: Dwelling floated from its foundation and stranded in corn field over a mile away when flood receded.*

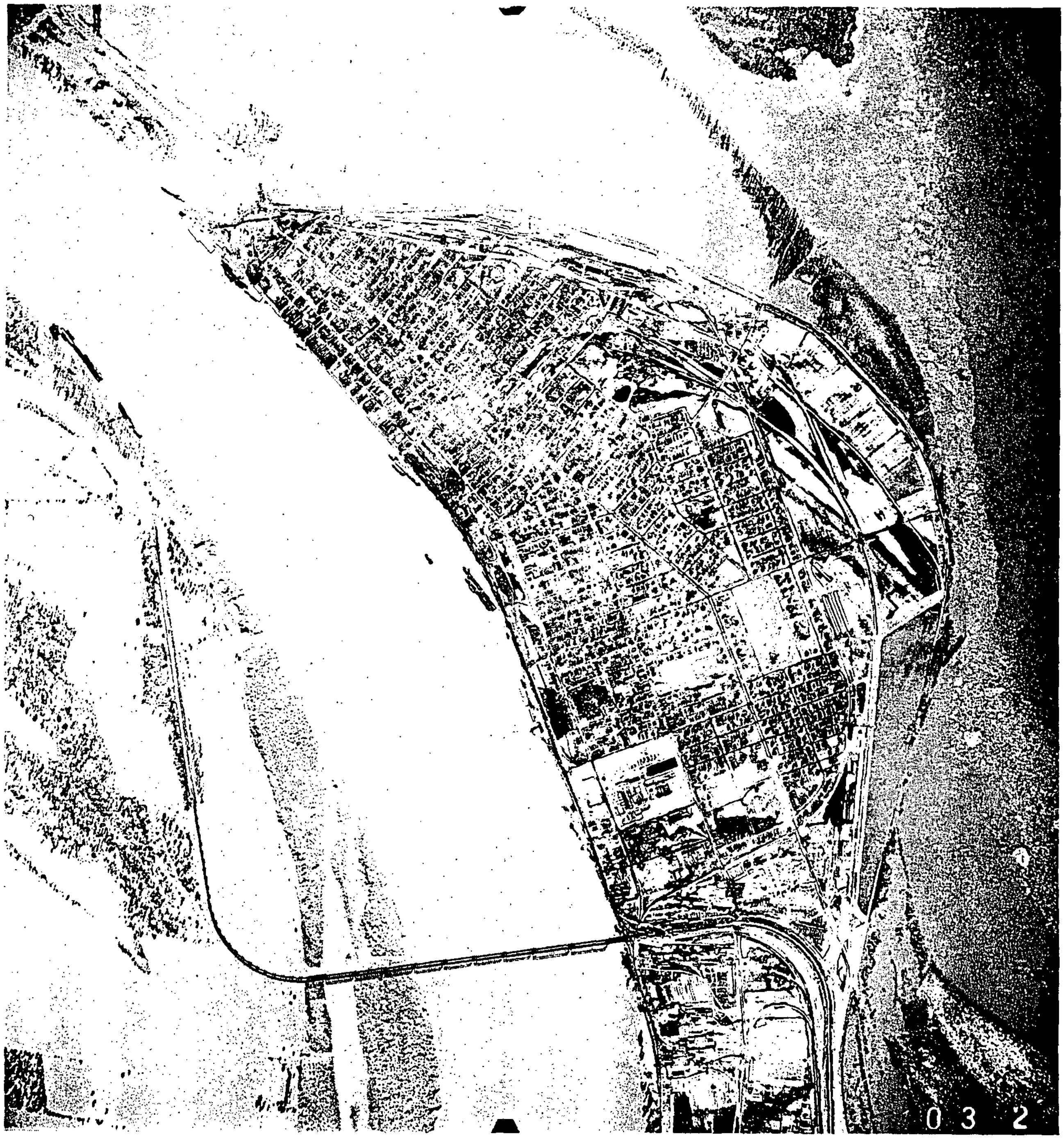


FIGURE 82.—*Cairo, Illinois, surrounded by 1937 flood. Had the Birds Point-New Madrid floodway not been opened, Cairo would have been completely submerged to a depth of over 20 feet. (Ohio River at left and Mississippi River at top and right—both rivers flow toward upper left corner in this view.)*

statute serves the purpose of integrating flood control operations of the Kentucky Reservoir with the flows on the Ohio and Mississippi Rivers under the control of the Corps of Engineers. The Division Engineer, Ohio River Division, has been designated formally by the Secretary of War as the representative of the War Department, responsible for issuance of instructions to TVA for regulating releases from the Tennessee River when danger from floods exists on the lower Ohio and Mississippi Rivers.

Since the above statute was enacted, periodic conferences have been held between the Corps of Engineers and TVA engineers to perfect arrangements by which the exchanges of flood data are made and to clarify matters pertaining to the release of water from the Tennessee River in time of flood. The results of these conferences were formalized by a joint manual [1] which was issued in 1957. It was agreed that when the Cairo stage is 35 feet, with 40 feet or higher predicted, TVA shall report the observed data for mainstream reservoirs from Chickamauga to Kentucky Dams, inclusive, by teletype early each day to the Division Engineer, Ohio River Division at Cincinnati, giving rainfall, stages, and discharges. Later each day, predicted schedules of stages and discharges from three to five days in advance are reported. The Ohio River Division office of the Corps of Engineers at Cincinnati reciprocates by furnishing to TVA observed data and predicted discharges at a number of key points on the Ohio and upper Mississippi Rivers and on certain tributaries. In addition, when the Cairo stage is 50 feet or higher, that office relays data daily from the Mississippi River Commission at Vicksburg to TVA, which includes observed stages and predicted stages on the Mississippi River from Cairo to New Orleans six days in advance, together with prediction of date and height of crest at all stations.

Criteria for determination of existence of floods—The following criteria for determination of the existence of danger of floods on the lower Ohio and Mississippi Rivers are established in the joint manual: [2]

a. Danger from floods in the lower Ohio and Mississippi Rivers will be considered to exist whenever a stage of 44 feet on the Cairo gage is predicted, and
b. Whenever flood conditions below Cairo are such that releases from the Tennessee River should be regulated to prevent aggravation thereof.

Critical flood stages—main control points—Concerning critical flood stages and main control points for the lower Ohio and Mississippi floods the joint manual [3] contains the following:

> The primary objectives in the operation of reservoirs for regulation of floods on the lower Mississippi River are (1) to safeguard the Mississippi River levee system; (2) to reduce the frequency with which the Birds Point-New Madrid floodway is put to use; and (3) to reduce the frequency of flooding of land not protected by the levee system. As a guide to regulation of releases from the Tennessee River to obtain the maximum feasible degree of flood protection, certain locations on the lower Mississippi River have been selected as main control points and critical stages determined for each location. In general, these critical stages are those above which the danger of floods breaching or overtopping protection works is imminent, or damage to unprotected areas becomes serious. The control points and the critical stages at these points are as follows:

Location	*Critical stage*
Cairo, Illinois	54 feet
	44 feet—cropping season—end of April to end of November
Birds Point-New Madrid Floodway	57 feet—Cairo gage
Tiptonville-Obion extension levee	52.5 feet—Cairo gage

> Flood stage at Cairo, Illinois, is 40 feet. Some flooding of highways and railroads occurs at stages below 50 feet and flooding becomes more extensive about 53 feet. However, studies of Kentucky Reservoir operation for floods of record indicate that 54 feet is the practical minimum stage to which the more severe winter season floods can be reduced with the storage available. Accordingly, a Cairo stage of 54 feet has been selected as the critical stage for winter season floods occurring during the period of January through March. With the beginning of the cropping season, about the first of April, lower stages cause damages by delaying preparation of agricultural lands and the planting of crops and by actual destruction of crops. While control of summer season floods to a stage of 40 feet would be desirable, analysis of past floods indicates 44 feet to be the minimum practical stage for reduction of severe summer season floods. Accordingly, the critical stage at Cairo for the cropping season period from the end of April to the end of November has been set at 44 feet.

This does not mean that all winter floods reaching a stage of 54 feet or that all summer floods reaching a stage of 44 feet will be regulated, nor does it preclude that under certain conditions floods lower than these stages will be regulated.

RESERVOIR OPERATION BY RULES

Fixed rule operation

In making a study of the potential effect of reservoir operation on the stages of floods downstream, it was essential that a method be evolved which would give comparable results in the study of different floods, approximate the results of actual operation, and at the same time, assure that the available storage is not filled before the peak of the flood is reached.

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1. *Regulation of Releases from the Tennessee River During Ohio and Mississippi River Flood Periods.* Prepared by U. S. Army Engineer Division, Ohio River, Corps of Engineers, Cincinnati, Ohio, and Tennessee Valley Authority, Knoxville, Tennessee, 1 August 1957.
2. Ibid., p. 2.
3. Ibid., pp. 13-15.

Such a method is encompassed in the "fixed rule" operation. This method is based on the premise that the space reserved for flood storage in a reservoir should be completely filled only when a flood approaching the maximum probable occurs or when the rainfall pattern causing a flood of major proportions has fully developed and has been largely dissipated. Of course, this applies only to reservoirs lacking sufficient capacity to store the maximum flood. Filling then involves relating the discharge from the dam at any time to the rising reservoir level at that time.

In any given actual operation, if followed in detail, this method would preclude varying the discharges from those specified by the rule so as to conform with unusual streamflow conditions existing in the main stream or with anticipated increases in streamflow from predicted rainfall. Nevertheless, when used as a guide, the "fixed rule" method is a useful tool for checking the operation of the reservoir, although it has not been applied in actual operation.

The application of the "fixed rule" to the prediction of reservoir operations would be based on anticipated rainfall and runoff. A setup would be made each day during the flood control operations for adherence to the "fixed rule" when applicable. On the following day, any difference between the rule and the actual reservoir stage and discharge could be corrected. This would result in a stepped line approximating the straight line shown on the charts.

Studies of fixed rule operation—Fixed rules have been developed for Watauga, South Holston, Cherokee, Douglas, Fontana, and Norris Reservoirs, and for Chatuge, Nottely, and Hiwassee combined. These rules are in the form of graphical relationships (fig. 83) which show the relation between headwater elevation and the rate of discharge which should prevail during the flood. The rules were designed to make full use of the tributary storage, except for the emergency reserves shown on the charts, in a flood like the Chattanooga design flood. They are intended for use during the major flood season of January, February, and March. Use of the rules would begin when it is recognized that a flood is developing. No foreknowledge of the streamflow is necessary. The use of the rules results in an operation similar to that of a detention basin, but retains the benefits of gate-controlled outlet works. Moreover, in comparing actual operations since 1946 with fixed-rule operations, the latter in every instance gives peak reductions as good or somewhat better than those actually attained.

The fixed rule for Fontana Reservoir (fig. 83) shows that as long as the headwater level is below elevation 1649 only discharges up to turbine capacitv may be made. Between headwater elevation 1649 and the maximum elevation reached, discharges are determined from the "Guide Curve—Filling." Thus, as the reservoir level rises and the remaining available storage becomes less, the discharge increases. Changes in gate settings may be made as often as necessary, but once every six hours should be sufficient. When the crest headwater elevation is reached, drawdown is accomplished by following the rules as indicated by the notes on the chart. The maximum discharge at Fontana would be 30,000 cubic feet per second unless the reservoir was filled to the specified elevation 1702, and inflows greater than this amount must be discharged. The drawdown rate of 15,000 cubic feet per second is about one-half the safe carrying capacity of the river channel on the lower Little Tennessee River. For study purposes, routing curves conforming to the fixed rule guides have been prepared so as to eliminate the trial and error steps otherwise necessary in following the guides.

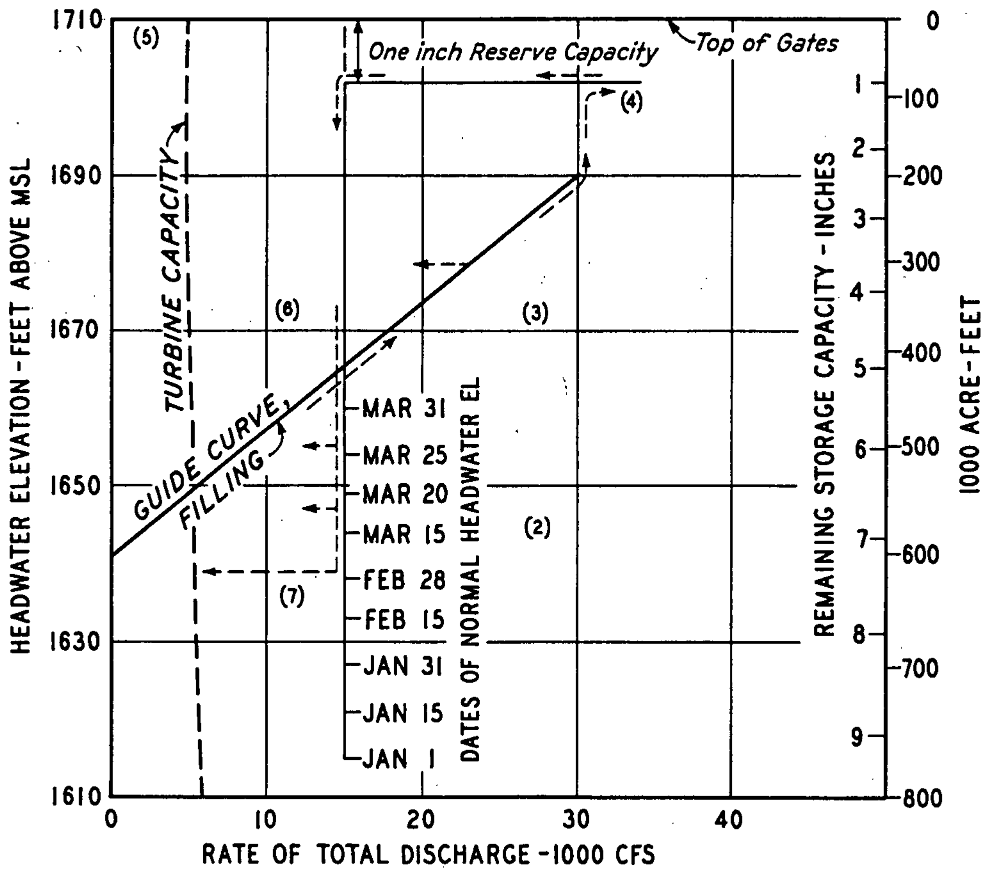
A reserve storage capacity equivalent to 1 inch depth over the drainage area in Norris, Hiwassee, and Fontana and ½ inch in Cherokee would not be filled except in the case of a subsequent storm occurring while the reservoirs were substantially full, or for additional storage for the benefit of the lower Tennessee, Ohio, and Mississippi Rivers.

The use of these charts for the tributary reservoirs and others for the main-river reservoirs would result in reducing the Chattanooga design flood by 17 feet to a 60-foot stage, and major reductions in all other damaging floods.

A typical example of tributary reservoir operation according to this fixed rule is given in table 24 for part of the 1867 flood period in Fontana Reservoir.

Assumptions made in this operation are that (1) the reservoir level at the beginning of the flood would be at elevation 1640, the normal for March 5, as shown in figure 83; (2) the inflow would be equal to that computed for the 1875 flood increased by 10 percent; (3) the turbine discharge would average 3,000 cubic feet per second; (4) the operating guide shown in figure 83 would be used during the flood; and (5) a routing interval of one day would be suitable.

This guide shows that with headwater elevation 1640 only turbine flow (assumed at 3,000 cubic feet per second) would be discharged. Storage corresponding to elevation 1640 on March 5 is 834,000 acre-feet, and this is tabulated for that date. The sum of inflows at the beginning and ending of the period from March 5 to March 6 and storage at the beginning of the period is tabulated and the initial discharge of 3,000 cubic feet per second is subtracted from this sum, giving the discharge plus storage factor for the end of that period of 932,000. Assuming a discharge at the end of the period of 7,000 cubic feet per second and subtracting this rate from 932,000 gives a storage of 925,000 acre-feet, which is tabulated for March 6. The corresponding headwater elevation of 1652.4 (fig. 84) indicates the discharge of 7,000 cubic feet per second was correctly assumed according to the guide (fig. 83) and subse-



(1) In case of flood, discharge not more than turbine capacity until guide curve elevation is reached. Should reservoir already be above the guide, do not lower headwater and do not exceed normal headwater until guide curve flow is reached. Thereafter, use guide curve with headwater elevations to determine total outflow rates, and follow notations on the drawing to determine emptying procedures.

Normal headwater is the maximum multiple-purpose level between January 1 and March 31.

Turbine capacity is the discharge required to develop generator capacity.

(2) For floods in which the highest outflow is less than 15,000 cfs, empty reservoir to normal elevation at a rate between this maximum and 15,000 cfs.

(3) For floods in which the highest outflow exceeds 15,000 cfs, hold the corresponding maximum elevation until discharge recedes to the emptying rate of 15,000 cfs.

(4) For floods in which elevation 1702 is reached, hold that elevation until discharge recedes to the emptying rate of 15,000 cfs.

(5) For floods in which elevation 1702 is exceeded because of insufficient outlet capacity, hold outlets open until elevation recedes to 1702, then hold that elevation until discharge recedes to the emptying rate of 15,000 cfs.

(6) Empty to normal headwater.

(7) Return to turbine discharge when normal headwater is reached.

FIGURE 83.—*Guide for operation during flood season—Fontana Reservoir.*

TABLE 24.—*Computation of Fontana Dam discharge—fixed-rule operation—1867 flood.*

	Unit	Date—Month of March															
		5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Inflow$_1$, I_1	1000 cfs	36	65	42	20	13	20	40	23	18	9	8	11	16	8	6	5
Inflow$_2$, I_2	1000 cfs	65	42	20	13	20	40	23	18	9	8	11	16	8	6	5	5
Storage$_1$, S_1	1000 ac-ft	834	925	1011	1043	1044	1045	1071	1096	1097	1089	1076	1065	1062	1056	1040	1021
$I_1 + I_2 + S_1$	1000's	935	1032	1073	1076	1077	1105	1134	1137	1124	1106	1095	1092	1086	1070	1051	1031
Discharge$_1$	1000 cfs	3	7	14	16	16	16	18	20	20	15	15	15	15	15	15	15
Discharge$_2$ + storage$_2$	1000's	932	1025	1059	1060	1051	1089	1116	1117	1104	1091	1080	1077	1071	1055	1036	1016
Headwater elevation$_1$	Feet	1640.0	1652.4	1663.4	1667.3	1667.4	1667.5	1670.6	1673.6	1673.7	1672.7	1671.2	1669.9	1669.6	1668.9	1666.9	1664.6

Note: Subscripts 1 and 2 refer to the beginning and the end of the time interval, respectively.

quent discharge rates may then be determined in a similar manner up to March 13. At this time the maximum elevation in this particular flood is reached, and the reservoir is emptied according to note (3) on figure 83, that is, at the rate of 15,000 cubic feet per second. This rate is continued until March 29 when the normal headwater is reached and the discharge is reduced to turbine flow. The computation in table 24 covers only the major portion of the flood period.

Similar fixed-rule charts were also prepared for the main Tennessee River reservoirs. These charts also show the relation between headwater elevation at each dam and the rate of discharge which should prevail during a flood. In the case of Chickamauga Reservoir, Chattanooga stage is substituted for dam discharge. Because of the limited storage capacity,

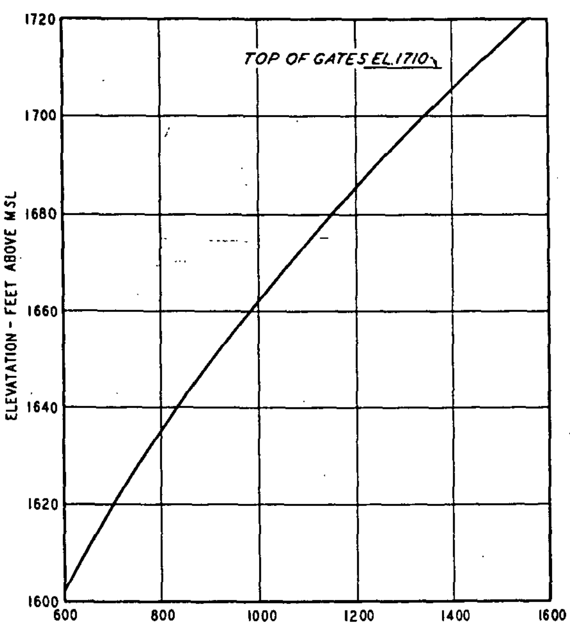


FIGURE 84.—*Reservoir volumes—Fontana.*

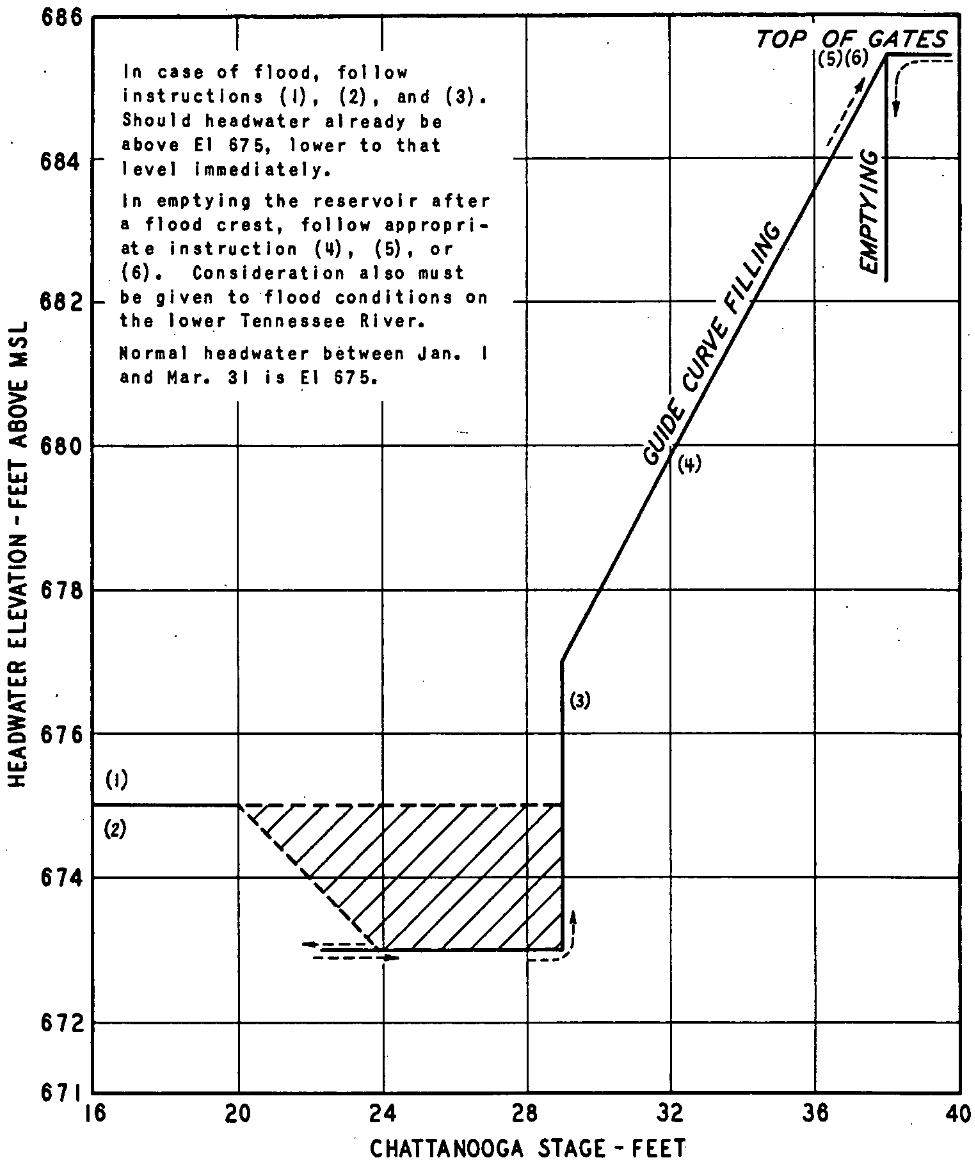
these rules were designed to make full use of the reservoir storage in a flood like the maximum known flood instead of the Chattanooga design flood.

The chart for Chickamauga (fig. 85) shows that in case of flood, normal headwater elevation 675 is held until the Chattanooga stage rises to 20 feet. When this stage is reached, the headwater level is drawn down to elevation 673 by increasing the outflow and allowing Chattanooga stage to rise to as high as 29 feet. If the flood continues, Chickamauga outflow is restricted to a 29-foot Chattanooga stage until headwater reaches elevation 677. Thereafter Chickamauga discharge and Chattanooga stage increase as the headwater rises. When the headwater level reaches a maximum, drawdown is accomplished by continuing the maximum Chattanooga stage reached or, if that rose above 38 feet, by allowing the stage to recede to 38 feet and then continuing the 38-foot stage. Drawdown after the flood depends, of course, on flood conditions in the lower river. If such drawdown would adversely affect lower river stages, it would be delayed unless another headwater flood develops. A computation of the fixed-rule operation of Chickamauga Reservoir in the 1867 flood is given in table 25.

Ideal operation

The so-called "ideal" method of operation of the reservoirs for flood control, where reservoirs do not have sufficient capacity to retain the entire runoff from a storm, implies the best possible use of the storage available for the maximum reduction of flood stages at critical locations below the reservoirs.

In order to accomplish this result, it would be necessary to have in advance of the storm complete knowledge of the discharge of the stream that would occur at various locations. Since any such degree of accuracy in forecasting does not appear to be attainable in the foreseeable future, there seems to be no point in anticipating such favorable results. Although it is true that the possibilities of this method of operation can be computed from records of past



(1) Hold El 675 until Chattanooga stage equals 20 feet.

(2) Lower headwater to El 673 by increasing stage to as high as 29'. If after reaching El 673, stage is below 29' but flood continues to develop upstream, hold this level until stage rises again to 29'. If flood does not develop, return headwater to normal.

(3) Hold 29-foot stage until headwater El 677 is reached. Thereafter, use guide curve with headwater elevations to determine Chattanooga stage.

(4) For floods in which the highest elevation reached is below top of gates, lower headwater to normal by continuing the crest Chattanooga stage.

(5) For floods in which top of gates is reached, hold that elevation until Chattanooga stage recedes to 38 feet. Then lower headwater to normal by continuing the 38-foot stage.

(6) For floods in which the spillway capacity at gate top level is exceeded, and the headwater rises above that level, continue discharging at capacity until headwater returns to top of gates. Hold that elevation until Chattanooga stage recedes to 38 feet. Then lower headwater to normal by continuing the 38-foot stage.

FIGURE 85.—*Guide for operation during flood season—Chickamauga Reservoir.*

TABLE 25.—*Computation of Chickamauga Dam discharge—fixed-rule operation—1867 flood.*

	Unit	Date—Month of March															
		5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Watts Bar Dam discharge	1000 cfs	70	100	119	216	162	159	163	214	210	200	176	148	154	161	140	140
Hiwassee Dam discharge[a]	1000 cfs	2	2	4	8	10	10	11	12	13	14	14	10	10	10	10	10
Chickamauga local[b]	1000 cfs	9	43	104	59	27	6	11	54	51	35	18	13	9	21	19	10
Inflow$_1$, I_1	1000 cfs	81	145	227	283	199	175	185	280	274	249	208	171	173	192	169	160
Inflow $_2$, I_2	1000 cfs	145	227	283	199	175	185	280	274	249	208	171	173	192	169	160	—
Storage$_1$, S_1	1000 ac-ft	407	402	464	629	712	661	607	665	790	845	815	706	570	507	497	477
$I_1 + I_2 + S_1$	1000's	633	774	974	1111	1086	1021	1072	1219	1313	1302	1194	1050	935	868	826	—
Chickamauga discharge$_1$	1000 cfs	81	150	160	185	214	211	203	204	225	243	244	244	236	192	179	170
Discharge$_2$ + storage$_2$	1000 cfs	552	624	814	926	872	810	869	1015	1088	1059	950	806	699	676	647	—
Headwater elevation$_1$	Feet	675.0	673.0	674.0	678.9	681.7	680.7	679.1	680.3	683.6	685.1	684.4	681.1	675.5	675.0	675.0	675.0
Chattanooga local$_1$[c]	1000 cfs	13	23	13	6	2	5	13	12	8	4	3	3	5	4	2	2
Chattanooga discharge$_1$	1000 cfs	94	173	173	191	216	216	216	216	233	247	247	247	241	196	181	172
Chattanooga stage$_1$	Feet	20.0	29.0	29.0	31.0	34.0	34.0	34.0	34.0	36.0	37.6	37.6	37.6	36.9	31.7	29.9	28.9

Note: Subscripts 1 and 2 refer to the beginning and the end of the time interval respectively.
a. One-day-lag.
b. Discharge from the drainage area between Watts Bar and Chickamauga Dams, excluding the area above Hiwassee Dam.
c. Discharge from the drainage area between Chickamauga Dam and the Chattanooga Walnut Street stream gage.

floods in order to determine the limiting reductions of stage, the results are apt to be misinterpreted as those which should have been obtained but were not because of incompetence of the operators.

COORDINATION OF FLOOD CONTROL WITH OTHER PURPOSES

As previously cited, the TVA Act provides that the reservoirs are to be operated primarily for navigation and flood control, and for generation of power so far as consistent with these two purposes. No other purposes are specified as to operation of reservoirs, and therefore any other purpose for which they may be used must be consistent with navigation, flood control, and power. The principal other use of the reservoirs is for recreation. They are also fluctuated during the summer season to avoid increase in the production of malaria-bearing mosquitoes.

The priority assigned to various purposes by law is strictly observed. Although this results in somewhat less than ideal conditions for secondary purposes, as described on the following pages, the present reservoir conditions are generally so superior to previous natural conditions that the resolution of conflicts in this manner can hardly be looked upon as involving a sacrifice.

Maintenance of navigation pool levels

Pursuant to the comprehensive Tennessee River Survey report contained in House Document No. 328, 71st Congress, 2d Session, the River and Harbor Act of July 1930 authorized a 9-foot navigation project from the mouth of the Tennessee River to Knoxville, Tennessee, a river distance of about 650 miles. The subsequent TVA Act, approved May 18, 1933, directed TVA, among other things, to improve navigation in the Tennessee River and its tributaries. In complying with this mandate, the dams on the main Tennessee River were located and designed to provide a chain of slack-water navigation pools, with minimum flat pool levels so established that they would afford, upon completion of supplementary dredging in the upper ends of the pools, the channel width and depth required for 9-foot-draft navigation (fig. 86). The drawdown permitted in flood control operations below these levels can only be made when backwater slopes during high flows are sufficient to afford adequate channel depths. At these levels, a depth of at least 11 feet is available along the sailing line throughout the pool and at least 10 feet over the lock sills.

Power generation reconciled with primary purposes

In planning each of the reservoirs the requirements of navigation and flood control were the basic controlling features in the preparation of operating guides. Without this priority of navigation and flood control, it would be possible to operate the reservoirs to increase somewhat the generation of power. This increase of power could be effected largely through the retention of water in storage above the guide curve during the flood season.

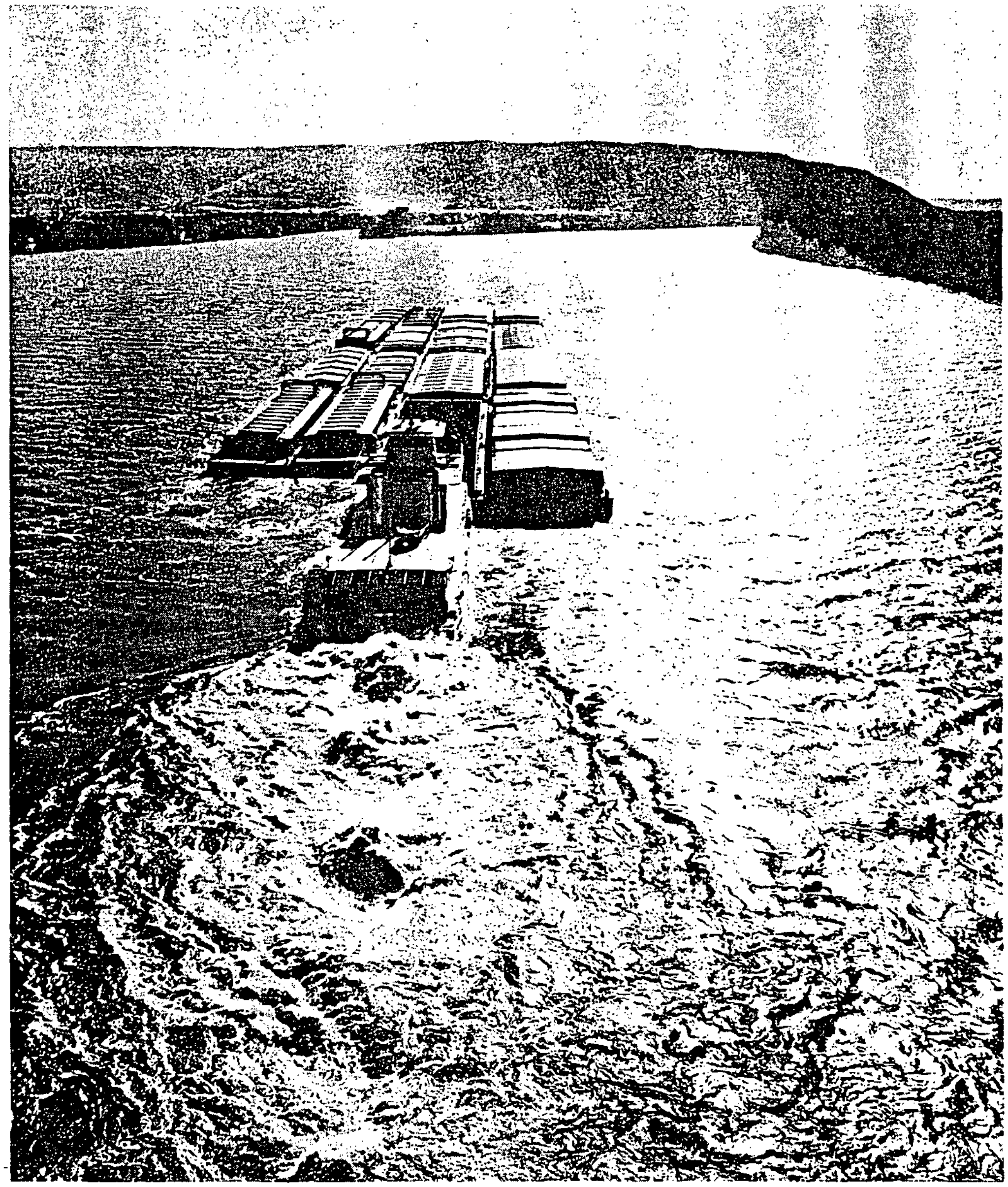


FIGURE 86.—*Commercial navigation on Wheeler Reservoir. In addition to flood control the main-river reservoirs provide slack-water navigation pools.*

There have been many occasions when all or part of such excess water, if retained, could have been used later in the generation of hydro power. A good example of TVA system operations during such an occasion was in the late fall of 1945. At that time, in order to draw the reservoirs down to the flood season level by January 1, a total of more than a million acre-feet of water in excess of turbine use was discharged from the five major tributary reservoirs (fig. 87) and through the mainstream reservoirs, augmented by an additional spill from the latter (fig. 95, page 158). Another example was in January and February 1946 when a large volume of water was spilled from the same tributary reservoirs following the regulation of two floods. Because of a subsequent dry spring and the resulting failure to accomplish the desired filling in these reservoirs, much, if not all, of this surplus water could have been impounded and used later to produce additional power at tributary and mainstream hydro plants.

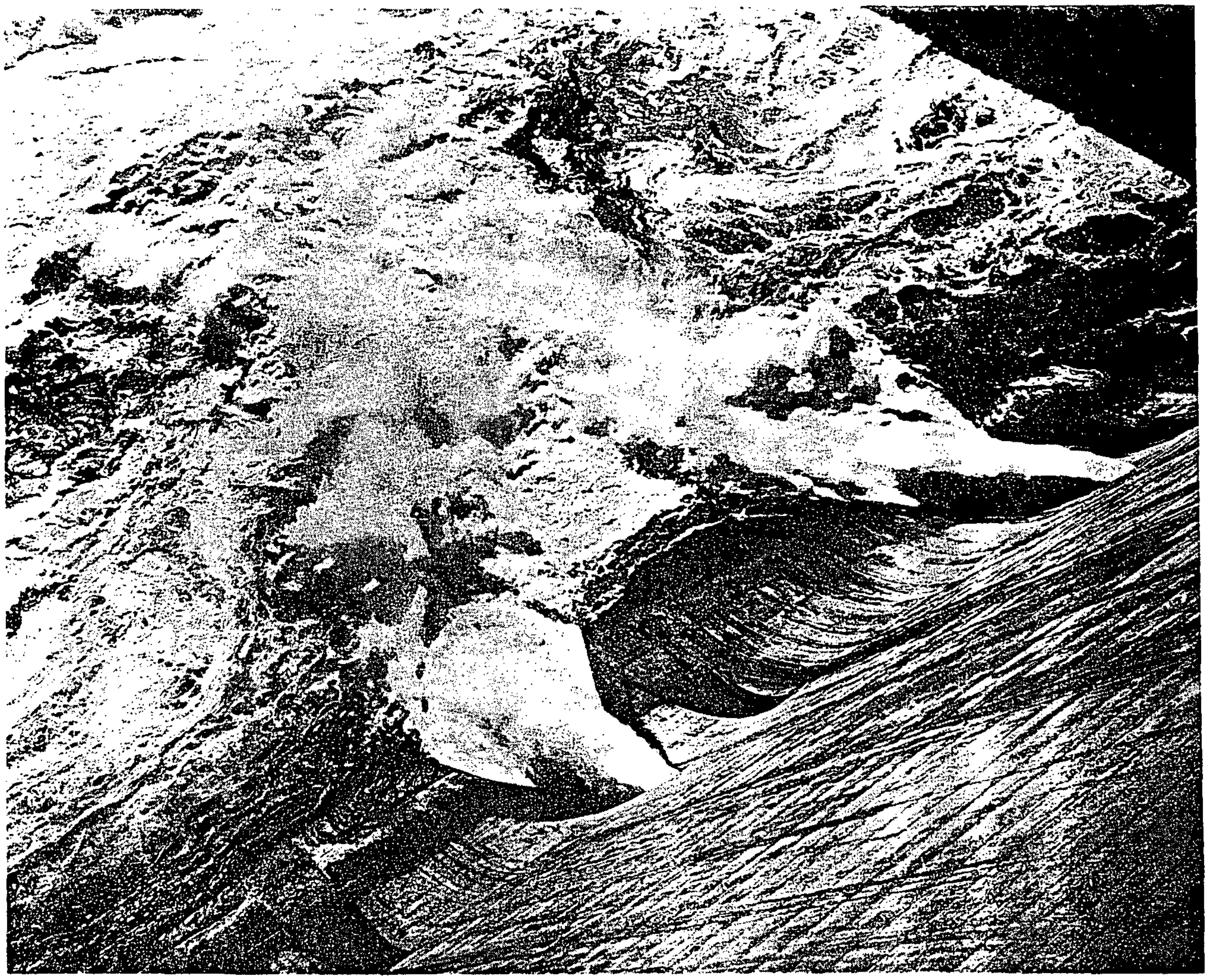


FIGURE 87.—*Tributary flood control dam discharging water in excess of turbine use to bring reservoir down to flood season level by January 1.*

Figure 113, page 179, is a chart showing the operation of Douglas Reservoir during that period and illustrates the extent of the loss of power generation due to this type of multiple-purpose operation.

It is clear that in coordinating power generation with flood control in the operation of reservoirs, the flood control priority permits less power output than otherwise, but even so there may be more power produced than would be possible in a power project of such size that it could be justified by power alone. In addition, flood control operations are sometimes actually beneficial to power generation, as will appear from the following description of the effect of such operations on the loss of head at mainstream plants during floods.

The early winter floods which occurred in 1946, 1947, and 1948 caused considerable reduction of generating capability due to high tailwater at mainstream plants before the storage of floodwater at several tributary plants had restored the head lost because of their previous seasonal drawdown. Both types of losses are the result of loss of head materially below that at which the various generating units are rated for full output. When major floods occur later in the winter, after recovery of head at tributary projects, the loss of power capability on the system is not so great because the two losses are not coincident. However, the regulatory effect of tributary storage reservoirs substantially reduces the tailwater stage during floods, and the loss of capability of mainstream plants is materially less than it would be without regulation. Also, such losses would be greater if it were not for the routing of major floods through the system.

FIGURE 88.—*Sail boating on Chickamauga Lake—flood control reservoirs provide excellent recreation facilities.*

Malaria control[1]

As the Tennessee River Basin, particularly that part below Chattanooga, is located in a region of malaria prevalence, it has been essential, in connection with the impoundages of the TVA, to institute measures to control the propagation of malaria mosquitoes. TVA has had a definite part in the very successful reduction of malaria which, in some localities, amounts to a gradual eradication of the disease. Of the many measures employed by TVA for malaria control, the management of water levels was considered from the beginning to be the most important. It seeks to control the propagation of the malaria-bearing mosquito *(Anopheles quadrimaculatus)*, a preventive rather than a curative measure. The difficulty was to fit the necessary manipulation of water levels into the primary operating purposes, and then to improve techniques.

Water level management for malaria control, as reflected in reservoir operations, attempts to (1) create and maintain a shoreline environment unfavorable for the laying of eggs, and (2) provide a varying water level favorable to stranding and subsequent destruction of the mosquito larvae. Thus, water level management for the control of mosquito breeding consists of the following reservoir operations, either singly or in combination: (1) initial seasonal filling, (2) surcharge, (3) constant level, (4) fluctuation, and (5) seasonal recession.

1. This subject is thoroughly treated in a manual prepared by the U. S. Public Health Service and Tennessee Valley Authority and published in 1947, entitled "Malaria Control on Impounded Waters."

This method of operation does not interfere with the regulation of the usually short and relatively small summer floods and fits well into the annual cycle of reservoir drawdown to provide adequate flood storage space during the flood season. It does not conflict with that primary operating purpose.

Recreation

Although recreation may not be included among the major operating purposes of the reservoirs, its importance has always been recognized. As the number of reservoirs increased, recreational use of the impounded lakes and other streams and surrounding areas grew rapidly. Camping, boating, fishing, hunting, swimming, and other outdoor activities have reached large proportions, far beyond the expectations of many (fig. 88). Regulation of reservoir levels for flood control, involving rapid fluctuations and in most tributary reservoirs extreme seasonal drawdowns, reduces to some extent the desirability of the reservoirs for recreational purposes. Nevertheless, recreation has grown to these proportions without sacrifice of the essential operations for flood control.

The seasonal pattern of highest levels during the height of the recreation season is favorable. In addition, reservoirs are operated, whenever praticable, to give suitable conditions for the development of recreational facilities. For example, every effort is made to encourage an increase of fish population by avoiding excessive drawdown in the water level of any reservoir in the spring during the fish spawning period, which occurs usually in April or May, depending upon the season and water temperatures, and lasts from two to four weeks.

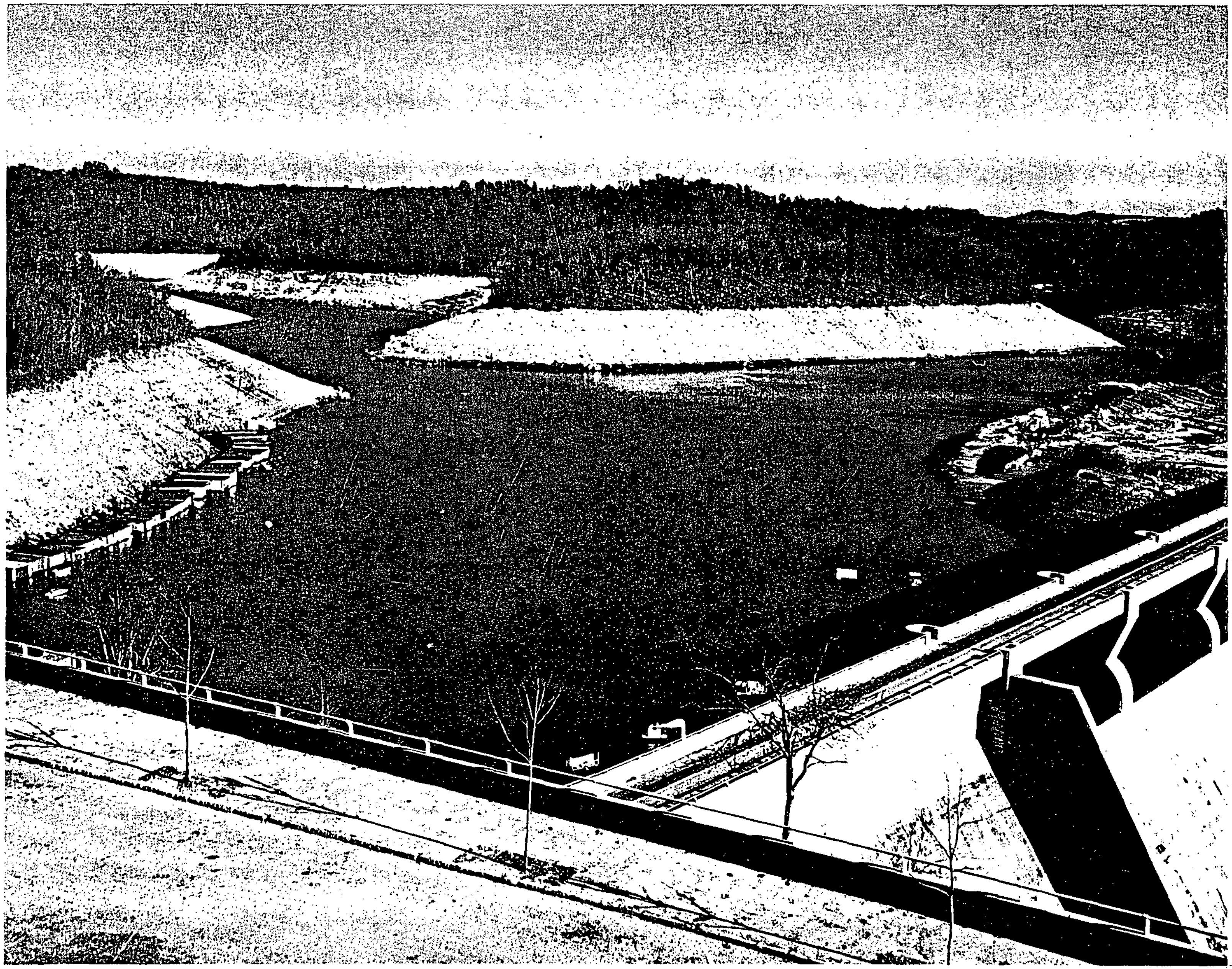


FIGURE 89.—Storage space available for flood control—Norris Reservoir, January 1956

CHAPTER 8

FLOOD CONTROL STORAGE AND ITS USE

Following a brief summary of the projects in the integrated system, this chapter discusses flood storage capacity required and its distribution above and below Chattanooga, including that proposed for Asheville and other areas along the upper French Broad River. The discussions then turn to outlet works capacity after which descriptions of main river and tributary reservoir functions conclude the chapter.

The integrated water control system in the Tennessee River Basin consists of 26 major dams and reservoirs built or acquired by TVA, 9 on the main river and 17 on the tributaries—construction of one of these 17 (Melton Hill) is still underway with completion scheduled for 1963. In addition, there are 6 major projects belonging to the Aluminum Company of America in the tributary Little Tennessee River Basin which are operated in accordance with instructions by TVA under agreement between the two parties. Of these 32 major projects, 8 of the 9 on the main river have a storage reservation for flood control; and of the 17 on the tributaries (not including those belonging to the Aluminum Company), 10 have a storage reservation for flood control. Figure 3 is a diagram of this system.

Thus, in the Tennessee Valley there are 18 projects where capacity for storing flood water is assured. These 18 together with Hales Bar, an acquired main river project, comprise the 19-project flood control system. Hales Bar is included in this system because of its beneficial regulating effect during most floods, although it was not feasible to provide a flood storage reservation when TVA renovated the dam. Drainage areas and flood storage reservation volumes of the 18 projects with such reservations are given in table 26, and their locations—together with that of Hales Bar and the other TVA reservoirs in the Valley—are shown in figure 90.

Statistical data covering the principal features of all 32 major water control projects in the Valley are given in table 1, page 6, and their locations are shown in figure 1, page 2.

In addition to the 32 major dams there are 13 minor dams in the Tennessee River Basin—3 acquired by TVA and 10 belonging to the Aluminum Company—which contribute power to the overall system.

FLOOD STORAGE CAPACITY REQUIRED

Although there are seven major projects (not including those owned by the Aluminum Company) which have no flood storage reservations, some of these will automatically have a beneficial regulating effect during most floods. Some of these reservoirs will be drawn down during the normally dry autumn for power generation and will be at low levels at the beginning of the flood season about the end of December. Subsequent filling during a flood would result in reduced flows. However, because in a prolonged wet period in January and February they might be filled or nearly filled by March, these reservoirs are not included as part of the dependable flood control system.

Other reservoirs have been and are being planned for addition to the system, including a group of seven detention-type reservoirs in the French Broad Basin above Asheville, North Carolina, for the control of floods at that city and other communities in the upper French Broad Basin and on farm land and industrial sites adjacent to the streams. This latter group has not been authorized.

The natural division of the Tennessee River Basin into two approximately equal parts having different physical characteristics made the planning of the reservoir system two more or less independent problems. Moreover, the principal points or areas subject to damaging floods are located at the downstream end of each of these two parts. At the downstream end in the eastern part, the city of Chattanooga presents one of the most serious urban flooding problems in the country. At and below the downstream end of the western part, the flooding of extensive agricultural lands, as well as cities, on the lower Ohio and Mississippi Rivers is of equal importance. Other flood problems of local importance, such as at Elizabethton, Kingsport, and Knoxville, also influenced the reservoir planning with respect to both the selection of reservoir sites and the capacity and operation of the reservoirs.

Distribution of storage above Chattanooga

The reservoir system in the eastern portion of the Basin, except for Boone, Watauga, and South Holston Reservoirs, was planned largely for the protection of the city of Chattanooga. This system also gives almost complete protection to large areas below the tributary dams and a substantial degree of protection to the areas below the main-river dams, which is particularly valuable in both areas during the crop growing season.

An ideal flood control plan would be one which would reduce all floods to a level not exceeding the

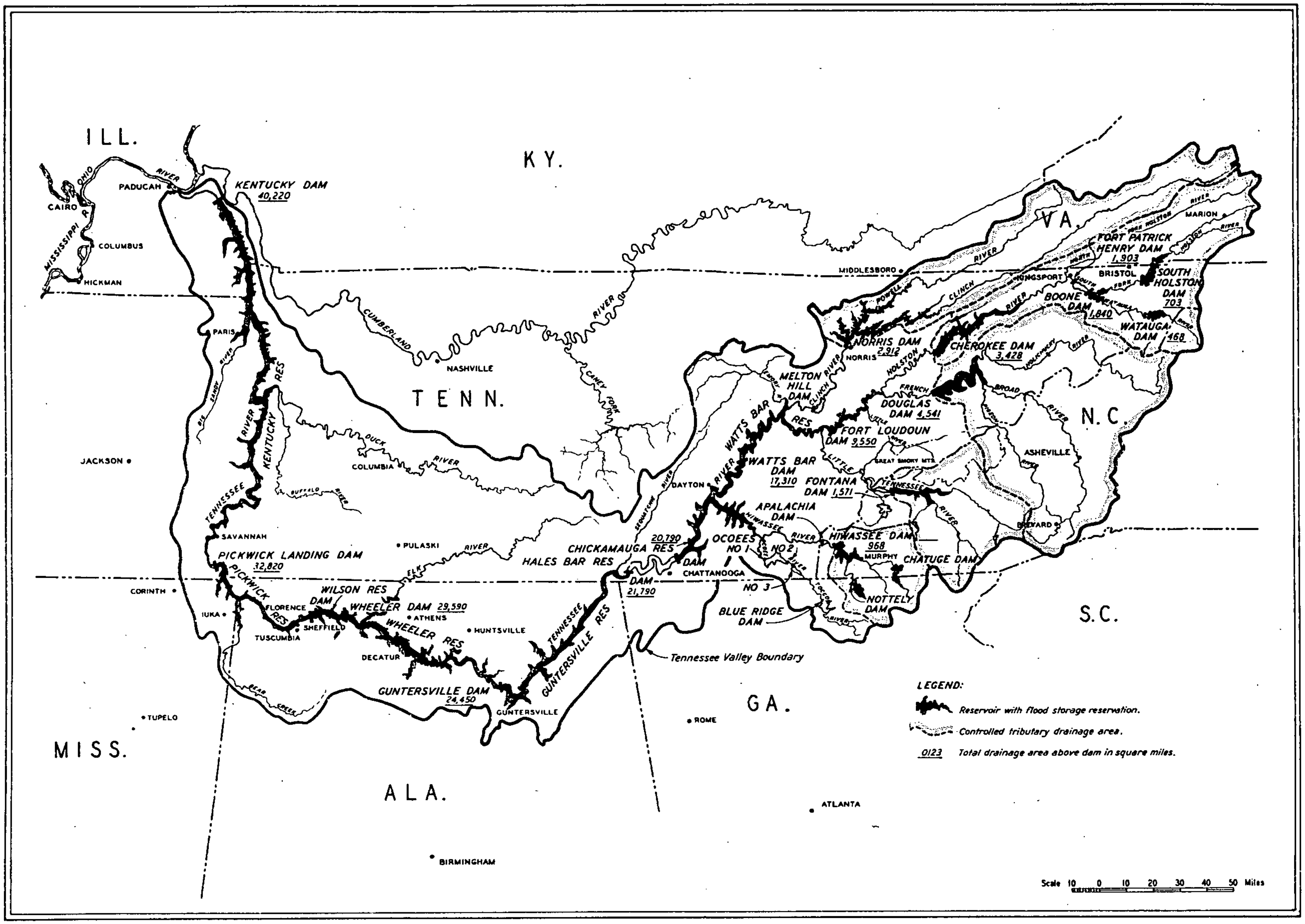


FIGURE 90.—*Flood control and other TVA reservoirs in the Tennessee Valley System.*

TABLE 26.—*Storage reserved for flood control.*

Reservoir	Drainage area above dam Total sq. mi.	Net[1] sq. mi.	Controlled H W Max El.	January 1 Seasonal El.	January 1 Storage[2] Ac.-ft	January 1 Storage[2] In.[3]	March 15 Seasonal El.	March 15 Storage[2] Ac.-ft	March 15 Storage[2] In.[3]	Summer level Seasonal El.	Summer level Storage[2] Ac.-ft	Summer level Storage[2] In.[3]
Tennessee River Projects:												
Kentucky	40,200	7,380	375[4]	354	4,010,800	10.19	354	4,010,800	10.19	359	1,044,200	2.66
Pickwick Landing	32,820	2,070	418	408	418,400	3.79	408	418,400	3.79	414	179,200	1.63
Wilson	30,750	1,160	507.88	504.5	53,000	0.85	504.5	53,000	0.85	507.5	6,000	0.10
Wheeler	29,590	5,140	556.28	550	347,500	1.27	550	347,500	1.27	556	19,400	0.07
Guntersville	24,450	3,660	595.44	593	162,900	0.83	593	162,900	0.83	595	31,200	0.16
Chickamauga	20,790	2,512	685.44	675	329,400	2.46	675	329,400	2.46	682.5	108,500	0.81
Watts Bar	17,310	3,277	745	735	377,600	2.16	735	377,600	2.16	741	163,500	0.94
Fort Loudoun	9,550	1,581	815	807	109,300	1.30	807	109,300	1.30	813	30,000	0.36
Total		26,780			5,808,900	4.07		5,808,900	4.07		1,582,000	1.11
Tributary Projects												
Hiwassee	968	565	1526.5	1455	291,100	9.66	1472	245,100	8.13	1524.5	12,400	0.41
Chatuge	189	189	1928	1910	105,400	10.46	1916	75,100	7.45	1927	7,100	0.71
Nottely	214	214	1780	1743	110,000	9.64	1755	83,500	7.32	1779	4,200	0.37
Norris	2,912	2,912	1034	978	1,635,000	10.53	990	1,377,000	8.87	1020	520,000	3.35
Fontana	1,571	1,571	1710	1615	771,200	9.20	1644	581,800	6.94	1708	21,200	0.25
Douglas	4,541	4,541	1002	935	1,311,200	5.41	958	1,019,800	4.21	1000	62,100	0.26
Cherokee	3,428	1,588	1075	1020	1,145,900	13.52	1042	807,200	9.52	1073	61,400	0.72
S. Holston	703	703	1742	1702	286,300	7.64	1713	218,200	5.82	1729	105,800	2.83
Boone	1,840	669	1385	1358	93,000	2.61	1374.6	41,400	1.16	1385	0	0
Watauga	468	468	1975	1934	256,200	10.26	1951.6	155,400	6.23	1959	109,000	4.37
Total		13,420			6,005,300	8.39		4,604,500	6.43		903,200	1.26
Grand Total	40,200				11,814,200	5.51		10,413,400	4.86		2,485,200	1.16
Total Above Chattanooga	21,400				6,821,600	5.98		5,420,800	4.75		1,205,200	1.06

1. Net drainage area excludes those areas above upstream storage dams.
2. Level pool storage between normal maximum elevation and seasonal elevation.
3. Equivalent depth over net drainage area.
4. Limited to elevation 365 for six months after June 1.

damaging stage, or lower. Such a plan would require that the proper amount of reservoir storage be distributed over the contributing area so that most of the inflow could be retained at the most advantageous time. Table 27 gives the flood volume above flood stage and above other stages at Chattanooga in several past floods and in the maximum probable flood. The volume of runoff above each stipulated stage is the least amount of storage which could reduce the flood to that stage. In the case of the main-river reservoirs, however, credit may be taken for the storage made available as a result of acceleration of the flood.

It is, of course, impossible always to make completely efficient use of storage space, partly because it is not probable that many storms would be distributed over the area in the exact manner as the reservoir storage is distributed. Consequently, to accomplish a desired degree of control, more storage must be provided than the minimum amount indicated in table 27.

One means of determining the amount of storage required to regulate the design flood at Chattanooga (maximum probable flood) is from a curve of cumulative or mass inflow, together with straight lines representing cumulative volume of discharge at uniform rates. Figure 91 shows plottings of the mass inflow of the Chattanooga maximum probable flood, as given in table 18, page 120, and of mass discharge for assumed rates of 300,000; 350,000; 400,000; 450,000; and 500,000 cubic feet per second. The greatest vertical distance between a mass discharge line in figure 91 and the mass inflow determines the

TABLE 27.—*Volumes in Chattanooga floods—unregulated.*

Flood	Crest stage, feet	Volume in units of 1000 acre-feet: Above flood stage of 30 feet	Above 40-foot stage	Above 50-foot stage	Above 60-foot stage
1867	57.9	3184	1840	656	—
1875	53.8	2460	1060	160	—
1886	52.2	2360	930	100	—
1917	47.7	1630	500	—	—
Maximum probable (design) flood	77.2	8224	6302	4272	2212

TABLE 28.—*Contribution of tributaries to Chattanooga floods in percentages of Chattanooga flood volumes.*

Tributary drainage area	Unit	Year of flood 1917	1918	1920	1926	1929	1932	1936	1946	1947	1948	1957	Eleven-flood average	Proportion of Chattanooga drainage area
Holston River above Cherokee Dam	Percent	12.4	14.9	9.5	13.7	8.7	10.1	12.4	10.2	11.8	11.7	15.8	11.9	16.0
French Broad River above Douglas Dam	Percent	11.4	14.2	14.8	10.4	12.7	16.1	17.5	15.5	13.9	13.8	14.9	14.1	21.2
Little Tennessee River above Fontana Dam	Percent	8.8	7.7	10.7	6.5	6.7	11.6	7.9	6.8	7.8	6.4	6.8	8.0	7.3
Clinch River above Norris Dam	Percent	17.5	19.9	9.2	21.1	15.8	11.5	12.6	12.6	13.4	13.7	14.8	14.8	13.6
Hiwassee River above Hiwassee Dam	Percent	4.7	3.7	6.3	2.5	3.7	6.5	4.9	4.1	4.5	3.7	4.2	4.4	4.5
Other	Percent	45.2	39.6	49.5	45.8	52.4	44.2	44.7	50.8	48.6	50.7	43.5	46.8	37.4
Total		100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Volume at Chattanogoa (100 percent)	1000 dsf	2780	2148	2654	2474	1683	1879	3875	2563	2672	2578	3375	—	—

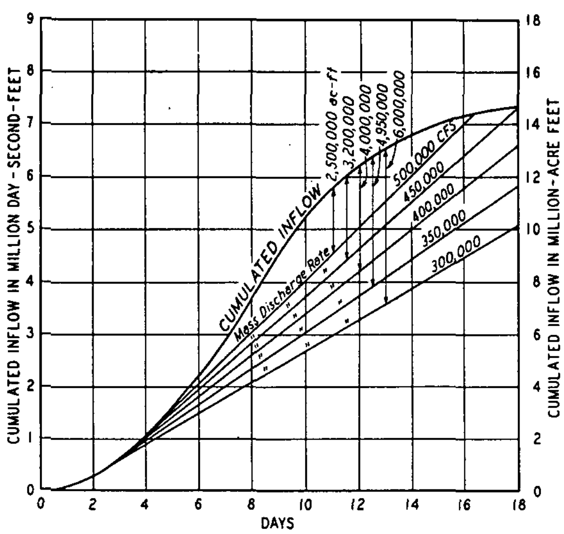


FIGURE 91.—*Mass curve of inflow—maximum probable flood at Chattanooga.*

storage required to reduce the maximum natural crest flow to that rate. To accomplish this reduction, storage would have to be located at proper points with respect to the flood-producing storm and would have to be utilized at the proper time with respect to the flood crest at Chattanooga.

The most economical and effective plan for complete protection against the design flood at Chattanooga was found to be a combination of reservoir storage and local protective works, principally earth levees. Although levees could be built lower or higher than the adopted stage, and a compensating greater or lesser storage might have been possible, consideration of all factors led to the adoption of a plan which called for not less than 4,000,000 acre-feet of flood storage and local protection works built for a flow line corresponding to 60 feet on the gage, or a flow of 486,000 cubic feet per second. A detailed discussion of all the factors affecting the selection of this stage is contained in the TVA report, "The Chattanooga Flood Control Problem," House Document No. 91, 76th Congress, 1st Session. This report pointed out that the 4,000,000 acre-feet of storage should be suitably distributed on the major tributaries, should be in addition to that in the main-river reservoirs, and should be available until the middle of March. Further studies also indicated that larger amounts were necessary at the beginning of the flood season about January 1.

Relative contributions in past floods—The five principal tributaries above Chattanooga drain a total of 18,640 square miles, of which 13,420 square miles are above the storage reservoirs. If all the tributaries contributed to floods at Chattanooga in proportion to their drainage areas, the 4,000,000 acre-feet of flood storage believed necessary would have been distributed in that proportion. This storage would be equivalent to 5.59 inches over the 13,420 square miles. As shown in table 28, however, some tributaries have contributed less and others more than the drainage area ratio; this is one of the reasons that the flood storage reservation in the tributary reservoirs is not a uniform amount per square mile. Flow contributions from the Clinch and Little Tennessee Rivers were generally more than in proportion with their drainage areas, from the Hiwassee River they were about equal to the drainage area ratio, and from the French Broad and Holston Rivers they were less. Table 29 shows the proportion of the total storage of 4,000,000 acre-feet which would be provided on each tributary on the basis of drainage area, on the basis of average contribution to Chattanooga floods, and the amount actually provided. The total amount provided (excluding that in Boone, Watauga, and South Holston Reservoirs) exceeds the 4,000,000 acre-feet by almost 200,000 acre-feet, and the actual distribution agrees more closely with the average tributary contribution than with the drainage area ratio.

Including Watauga, South Holston, and Boone Reservoirs, the storage available on March 15 for the entire Holston River area above Cherokee Dam is equal to 6.7 inches over the area. Thus, the control of this area is somewhat greater than that indicated as necessary by relative drainage areas or by average tributary contributions. The French Broad River Basin, however, is deficient in storage reserved for flood control because the limited storage in Douglas Reservoir provides the only economical control on that stream for Valley-wide floods.

Although the distribution of storage on the different areas should be based largely on the average contribution of the tributaries, other factors were also considered. For instance, the unit cost of storage in Norris Reservoir was less than in the other reservoirs, and consequently an amount even greater than was indicated by the average contribution was provided there.

Main-river reservoirs above Chattanooga—Because some physical limitation, such as a city, fixed the maximum water level, and requirements of navigation fixed the minimum levels, the three mainstream reservoirs above Chattanooga will store a comparatively small part of the total volume of a flood. But in spite of their small flood storage space, these reservoirs are essential for lowering the stage of the maximum probable flood to the point where local protective works will be feasible for the complete protection of Chattanooga because they provide regulation of the otherwise uncontrolled area of 7,980 square miles between Chattanooga and the tributary dams. As shown in table 26, the flood storage available in Fort Loudoun, Watts Bar, and Chickamauga Reservoirs, the three main-river projects above Chattanooga, is 1.30, 2.16, and 2.46 inches (equivalent depth over net drainage area), respectively. Although amounts comparable with the tributary storages would have been desirable, it was not practicable to secure them because of the physical obstructions in the reservoirs and navigation limitations. The small storage space in the main-river reservoirs must be preserved, insofar as possible, until the arrival of the flood crest and then filled to reduce the crest downstream. This operation is quite different from tributary reservoir operation, where relatively large volumes are available for storing floodwater. These operations are described in later pages.

Critical season—As shown by the seasonal distribution of floods at Chattanooga in figure 25, page 22, there have been no great floods between April 10 and December 20 in the 75 years since the gage was established. Although a few floods have occurred above flood stage of 30 feet during this April-December period, the highest, in November 1957, would have been less than 37 feet. Figure 25 also shows that more floods have occurred in March than in any other month. For these reasons, it appears valid to assume that although the maximum probable flood may occur in any year, it could occur only during the flood season from about January 1 to the following March 31; and with storage space sufficient to control this flood reserved until the middle of March, it was felt that even though the peak of a great flood occurred as late as April 1, it would begin as early as March 15, and the full amount of storage would be available at the beginning of the flood.

Reservoir storage below Chattanooga

A similar pattern of flood occurrences also is shown by the flood record at Johnsonville, except that the flood season is extended later into the month

TABLE 29.—*Comparative distributions among tributary reservoirs of tributary storage needed for Chattanooga flood control.*

	In proportion to				Adopted amount reserved on March 15	
	Drainage areas of tributaries		Average contributions of tributaries			
Reservoir	1000 ac-ft	Equiv. inches[1]	1000 ac-ft	Equiv. inches[1]	1000 ac-ft	Equiv. inches[1]
Cherokee	1020	5.59	895	4.90	807[2]	4.41[2]
Douglas	1350	5.59	1060	4.39	1020	4.21
Fontana	468	5.59	602	7.19	582	6.94
Norris	868	5.59	1112	7.17	1377	8.87
Hiwassee, Chatuge, Nottely	288	5.59	331	6.42	404	7.82
Total	4000[3]	5.59	4000[3]	5.59	4190	5.84

1. Depth over drainage area.
2. Excluding storage in Watauga, South Holston, and Boone Reservoirs.
3. Minimum amount recommended for reservation on March 15.

of April. The seasonal flood distributions at Paducah on the Ohio River and Cairo at the junction of the Ohio and Mississippi Rivers are also important because they influence the seasonal operation of Kentucky Reservoir. At both Paducah and Cairo, floods occur still later than on the Tennessee River, extending through April into May and June. If these late floods have their principal source largely outside the Tennessee River Basin, little regulation can be effected by the TVA reservoirs.

The principal purpose of flood control storage in the main river reservoirs below Chattanooga is (1) for the regulation of floods below each of the dams on the Tennessee River, and (2) on the lower Ohio and Mississippi Rivers. Reduction of discharge from Kentucky Reservoir is effective on the Ohio and Mississippi Rivers almost without diminution because of favorable location and availability of flood storage space. The flood storage space in the other reservoirs below Chattanooga (Guntersville, Wheeler, Wilson, and Pickwick) is extremely limited because of physical limitation on maximum levels and of navigation requirements fixing minimum levels.

Contribution of Tennessee River to Mississippi River floods—As shown in table 30, the contribution of the Tennessee River to Mississippi River floods is nearly always a substantial amount, although the drainage area of the Tennessee River Basin is only slightly more than 4 percent of the Mississippi River Basin above Columbus, Kentucky. This analysis of floods, occurring since 1897 and before appreciable reservoir regulation, for which there might have been a flood control operation in Kentucky Reservoir shows that the flow in the Tennessee River one day before the crest at Cairo, Illinois, was always a greater part of the Mississippi flow than indicated by the ratio of the drainage areas. The highest contribution, 30.6 percent, occurred in the 1897 flood, which was the highest known flood on the lower Tennessee River. On the average, the lowest Tennessee contributions are during those large Mississippi River floods which occur in the late spring and early summer after the Tennessee River flood season.

Kentucky Reservoir storage—The possibility of obtaining a 2- or 3-foot reduction in Cairo stages to supplement levees is also shown in table 30. Volumes in the top 2 and 3 feet in Mississippi River floods are compared with volumes in Tennessee River floods during a time period of equal length, but with an allowance for water travel. With few exceptions, there was sufficient flow in the Tennessee River, if controlled, to effect the specified reduction.

It would be unnecessary, of course, to store the entire Tennessee River flow indicated in table 30 to

TABLE 30.—*Contribution of the Tennessee River to Mississippi River floods.*

Flood		Cairo		Tennessee River discharge one day earlier		Volume in top 2 feet of Cairo crest			Volume in top 3 feet of Cairo crest		
Month	Year	Crest stage, feet	Discharge, 1000 cfs	1000 cfs	Percent of Cairo	1000 dsf	Length of period, days	Tennessee River volume in corresponding period, 1000 dsf	1000 dsf	Length of period, days	Tennessee River volume in corresponding period, 1000 dsf
March	1897	51.7	1553	475	30.6	673	17	6417	2177	34	10,903
March	1903	50.57	1518	257	16.9	745	12	3120	1282	15	3,538
May	1908	45.1	1301	78	6.0	545	10	738	1123	12	878
April	1912	53.94	2015	272	13.5	2210	17	4311	3395	19	4,868
May	1912	49.2	1532	279	18.2	661	9	2305	1203	11	2,763
April	1913	54.69	2015	213	10.6	1877	14	2565	3250	16	2,912
February	1916	53.21	1724	131	7.6	1103	9	1243	1888	11	1,598
June	1917	44.5	1187	55	4.6	558	10	552	867	12	638
May	1920	49.5	1343	161	12.0	333	8	1183	571	11	1,478
March	1922	53.6	1503	207	13.8	500	11	2507	1045	14	3,033
April	1922	53.5	1508	133	8.8	510	18	2255	1027	20	2,542
April	1927	56.4	1765	205	11.6	888	10	2100	1337	12	2,535
June	1927	49.6	1343	94	7.0	343	7	625	777	10	785
July	1928	45.5	1236	123	10.0	1055	12	1543	1363	14	1,710
March	1929	51.8	1571	271	17.3	172	5	1280	302	6	1,615
April	1929	51.5	1565	276	17.6	145	5	1448	477	7	1,933
May	1929	52.7	1642	196	11.9	1013	15	2918	1557	19	3,597
April	1933	51.87	1353	184	13.6	562	8	1250	1005	10	1,578
May	1933	51.82	1336	136	10.2	793	12	1217	1355	14	1,435
May	1935	45.9	1050	78	7.4	505	10	738	800	12	885
April	1936	52.74	1390	330	23.7	1213	16	4765	1967	19	5,417
February	1937	59.51	2010	238	11.8	2908	16	4390	4308	18	4,825
May	1937	48.6	1210	207	17.1	385	6	1097	708	7	1,420

reduce the flood crest at Cairo by the specified 2 or 3 feet. It is not required, therefore, that storage space equal to those quantities be provided. As indicated by detailed routing studies, reductions of 2 or 3 feet were possible in the large floods by utilizing storage in Kentucky Reservoir to headwater elevation 375. It was for this reason that elevation 375 was adopted as the maximum controlled reservoir level.

Storage supplementing that of Kentucky—Storage in Guntersville, Wheeler, and Pickwick Reservoirs will supplement that in Kentucky Reservoir and thus aid in controlling floods on the Mississippi River. Their principal purpose, however, will be to regulate floods immediately below those dams. This is also true of flood control storage in Wilson Reservoir, but because of its extremely limited amount, not much regulation can be expected.

No flood control storage is provided in Hales Bar Reservoir because of physical limitations. Navigation requirements in this steep, narrow, gorge section of the river and the existence of the city of Chattanooga at the upper end prevented the enlargement of this reservoir to include flood storage reservations.

Infeasibility of tributary storage below Chattanooga—No flood control storage has been provided nor is any planned for the immediate future on tributaries of the Tennessee River below Chattanooga. Recent floods in 1946 and 1948 have called attention to local flood problems on the Elk River, particularly at Fayetteville, and on the Duck River at Columbia and Shelbyville. Reservoir storage on these streams would benefit these communities and, in the case of Elk River, would aid in the flood control operation of Wheeler Reservoir for the benefit of land below Wheeler, Wilson, and Pickwick Dams, but it was not economically feasible at the time this report was prepared.

Storage for Kingsport and Elizabethton

The occurrence of floods in both winter and summer on Watauga River at Elizabethton and on South Fork Holston River at Bluff City and Kingsport pointed out the need for substantial storage above these points throughout the year. Figure 73, page 110, shows the distribution of these floods at Kingsport. Although several of the largest floods have occurred in the summer months, those of May 1901 and August 1940 were outstanding. Many floods have occurred in other months and, in fact, more floods have occurred in the usual flood season from January to March, inclusive, than in the summer. Volumes of floods occurring in summer months generally are lower than in winter floods, and less storage capacity is therefore required during the summer season. Table 26 shows that a minimum storage equivalent to 2.83 inches over the net drainage area is provided in South Holston Reservoir and 4.37 inches in Watauga Reservoir. These amounts are sufficient to regulate the maximum known summer season floods to non-damaging stages and to give a substantial degree of regulation in maximum probable floods. Much greater amounts of storage (table 26) are provided during the winter months to care for floods of greater volume or for several floods occurring close together. Together these reservoirs will regulate the flow from 1,171 square miles above Kingsport, or 60.6 percent of the total area of 1,931 square miles. Watauga Reservoir will regulate 468 square miles of the area above Elizabethton, or 67.5 percent of the total area of 692 square miles.

Storage proposed for Asheville and other areas along the upper French Broad River

As at Kingsport and Elizabethton in the Holston Basin, floods occur at Asheville on the French Broad River in both winter and summer months. The greatest known flood, however, occurred in July 1916 following a storm which included one of the greatest 24-hour rainfalls known at that time. At Altapass, about 18 miles northeast of the French Broad Basin, a rainfall of more than 22 inches was reported in 24 hours. Reported rainfall in the Basin itself exceeded 16 inches at several stations, but flood heights indicate that even greater amounts occurred in certain local areas.

A design flood having a volume equivalent to 12 inches over the drainage area was adopted for planning the Asheville flood protection works. A system of seven detention type reservoirs was recommended which would reduce the stage of this flood to a point where it could be economically confined by a flood wall at Asheville. The amount of storage space in the seven reservoirs would vary according to effective time of travel between the reservoir site and Asheville. A storage equivalent of 10 inches would be available in the reservoir on Swannanoa River, the nearest to Asheville; 9 inches would be available in three others on Cane Creek, Clear Creek, and Mills River; 6 inches on Little and Davidson Rivers; and 4 inches at the site near Brevard on the upper reaches of the French Broad River. These storages are between the bottom of the detention basin and the spillway crest, but in extreme floods more storage would be used above the spillway crest. The seven reservoirs would control 489 square miles of a total of 945 square miles, or 52 percent of the drainage area above Asheville.

These dams would also reduce floods which cause severe damage to the fertile farm land and industrial sites lying along the upper French Broad River and its tributaries.

CAPACITY OF OUTLET WORKS

Table 16, page 114, gives the discharge capacity of spillway and sluice gates at each TVA project. Design details of the outlet works are given in the technical reports for the individual projects.

KENTUCKY DAM

SPILLWAY DISCHARGE

IN CUBIC FEET PER SECOND

TENNESSEE VALLEY AUTHORITY — HYDRAULIC DATA BRANCH

GATE ARRANGEMENT	HEADWATER ELEVATION																					GATE ARRANGEMENT
	354.0	354.1	354.2	354.3	354.4	354.5	354.6	354.7	354.8	354.9	355.0	355.1	355.2	355.3	355.4	355.5	355.6	355.7	355.8	355.9	356.0	
25	5,340	5,410	5,480	5,550	5,620	5,700	5,770	5,840	5,910	5,980	6,050	6,120	6,200	6,270	6,340	6,420	6,490	6,560	6,630	6,710	6,780	25
26	10,680	10,820	10,960	11,110	11,250	11,390	11,530	11,670	11,820	11,960	12,100	12,250	12,390	12,540	12,680	12,830	12,980	13,120	13,270	13,410	13,560	26
27	16,020	16,230	16,450	16,660	16,870	17,080	17,300	17,510	17,720	17,940	18,150	18,370	18,590	18,810	19,030	19,240	19,460	19,680	19,900	20,120	20,340	27
28	21,360	21,640	21,930	22,210	22,500	22,780	23,060	23,350	23,630	23,920	24,200	24,490	24,780	25,080	25,370	25,660	25,950	26,240	26,540	26,830	27,120	28
29	26,700	27,060	27,410	27,760	28,120	28,480	28,830	29,180	29,540	29,900	30,250	30,620	30,980	31,340	31,710	32,080	32,440	32,800	33,170	33,540	33,900	29
30	32,040	32,470	32,890	33,320	33,740	34,170	34,600	35,020	35,450	35,870	36,300	36,740	37,180	37,610	38,050	38,490	38,930	39,370	39,800	40,240	40,680	30
31	37,380	37,880	38,370	38,870	39,370	39,860	40,360	40,860	41,360	41,850	42,350	42,860	43,370	43,880	44,390	44,900	45,420	45,930	46,440	46,950	47,460	31
32	42,720	43,290	43,860	44,420	44,990	45,560	46,130	46,700	47,260	47,830	48,400	48,980	49,570	50,150	50,740	51,320	51,900	52,490	53,070	53,660	54,240	32
33	48,060	48,700	49,340	49,980	50,620	51,260	51,890	52,530	53,170	53,810	54,450	55,110	55,760	56,420	57,080	57,740	58,390	59,050	59,710	60,360	61,020	33
34	53,400	54,110	54,820	55,530	56,240	56,950	57,660	58,370	59,080	59,790	60,500	61,230	61,960	62,690	63,420	64,150	64,880	65,610	66,340	67,070	67,800	34
35	58,690	59,470	60,250	61,030	61,810	62,600	63,380	64,160	64,940	65,720	66,500	67,300	68,110	68,910	69,710	70,520	71,320	72,120	72,920	73,730	74,530	35
36	64,020	64,870	65,720	66,580	67,430	68,280	69,130	69,980	70,840	71,690	72,540	73,420	74,290	75,170	76,040	76,920	77,800	78,670	79,550	80,420	81,300	36
37	69,280	70,210	71,140	72,070	73,000	73,930	74,860	75,790	76,720	77,650	78,580	79,530	80,480	81,430	82,380	83,320	84,270	85,220	86,170	87,120	88,070	37
38	74,600	75,590	76,590	77,580	78,580	79,570	80,560	81,560	82,550	83,550	84,540	85,570	86,600	87,630	88,660	89,690	90,720	91,750	92,780	93,810	94,840	38
39	79,830	80,900	81,980	83,050	84,130	85,200	86,270	87,350	88,420	89,500	90,570	91,660	92,760	93,860	94,950	96,040	97,140	98,240	99,330	100,400	101,500	39
40	85,140	86,280	87,410	88,550	89,680	90,820	91,960	93,090	94,230	95,360	96,500	97,680	98,860	100,000	101,200	102,400	103,600	104,700	105,900	107,100	108,300	40
41	90,340	91,560	92,780	93,990	95,210	96,430	97,650	98,870	100,100	101,300	102,500	103,800	105,000	106,200	107,500	108,700	110,000	111,200	112,400	113,700	114,900	41
42	95,640	96,920	98,200	99,470	100,800	102,000	103,300	104,600	105,900	107,100	108,400	109,700	111,100	112,400	113,700	115,000	116,400	117,700	119,000	120,400	121,700	42
43	100,800	102,200	103,500	104,900	106,200	107,600	108,900	110,300	111,600	113,000	114,300	115,700	117,100	118,500	119,900	121,300	122,700	124,100	125,500	126,900	128,300	43
44	106,000	107,400	108,800	110,300	111,700	113,100	114,600	116,000	117,400	118,900	120,300	121,800	123,200	124,700	126,200	127,700	129,100	130,600	132,100	133,600	135,000	44
45	111,100	112,600	114,100	115,600	117,100	118,600	120,100	121,600	123,100	124,600	126,200	127,700	129,200	130,800	132,300	133,900	135,400	137,000	138,500	140,100	141,600	45
46	116,400	118,000	119,600	121,200	122,700	124,300	125,900	127,500	129,000	130,600	132,200	133,800	135,400	137,100	138,700	140,300	141,900	143,500	145,200	146,800	148,400	46
47	121,800	123,400	125,100	126,700	128,400	130,000	131,700	133,300	135,000	136,600	138,200	139,900	141,600	143,300	145,000	146,700	148,400	150,100	151,800	153,500	155,200	47
48	127,100	128,800	130,500	132,300	134,000	135,700	137,400	139,100	140,900	142,600	144,300	146,100	147,800	149,600	151,400	153,100	154,900	156,700	158,400	160,200	162,000	48
49	141,800	143,500	145,300	147,000	148,800	150,500	152,300	154,100	155,800	157,600	159,300	161,100	162,900	164,700	166,500	168,300	170,100	171,900	173,700	175,500	177,300	49
50	156,400	158,200	160,000	161,800	163,600	165,400	167,200	169,000	170,700	172,500	174,300	176,200	178,000	179,800	181,700	183,500	185,400	187,200	189,000	190,900	192,700	50
51	171,100	172,900	174,800	176,600	178,400	180,200	182,000	183,900	185,700	187,500	189,300	191,200	193,100	195,000	196,800	198,700	200,600	202,500	204,300	206,200	208,100	51
52	185,800	187,700	189,500	191,400	193,200	195,100	197,000	198,800	200,700	202,500	204,400	206,300	208,200	210,100	212,000	213,900	215,800	217,700	219,700	221,600	223,500	52
53	200,600	202,500	204,400	206,300	208,200	210,100	212,000	213,800	215,700	217,600	219,500	221,400	223,400	225,300	227,300	229,200	231,200	233,100	235,100	237,000	238,900	53
54	215,400	217,300	219,300	221,200	223,100	225,000	226,900	228,800	230,800	232,700	234,600	236,600	238,500	240,500	242,500	244,500	246,500	248,400	250,400	252,400	254,400	54
55	230,200	232,100	234,100	236,000	238,000	239,900	241,800	243,800	245,700	247,700	249,600	251,700	253,700	255,700	257,700	259,700	261,700	263,700	265,700	267,800	269,800	55
56	244,800	246,800	248,800	250,800	252,800	254,700	256,700	258,700	260,700	262,700	264,600	266,700	268,700	270,800	272,800	274,900	276,900	278,900	281,000	283,000	285,100	56
57	259,400	261,400	263,400	265,500	267,500	269,500	271,500	273,500	275,500	277,500	279,500	281,600	283,700	285,700	287,800	289,900	292,000	294,000	296,100	298,200	300,300	57
58	273,900	275,900	278,000	280,000	282,000	284,100	286,100	288,100	290,200	292,200	294,200	296,400	298,500	300,600	302,700	304,800	306,900	309,000	311,100	313,200	315,300	58
59	288,300	290,400	292,400	294,500	296,500	298,600	300,600	302,700	304,800	306,800	308,900	311,000	313,100	315,300	317,400	319,600	321,700	323,800	326,000	328,100	330,200	59
60	297,100	299,200	301,300	303,300	305,400	307,500	309,500	311,600	313,700	315,700	317,800	319,900	322,100	324,200	326,400	328,500	330,600	332,800	334,900	337,000	339,200	60
61	310,300	312,400	314,500	316,600	318,700	320,800	322,900	325,000	327,100	329,300	331,400	333,500	335,700	337,900	340,100	342,200	344,400	346,600	348,800	350,900	353,100	61
62	323,000	325,200	327,300	329,500	331,600	333,800	335,900	338,100	340,200	342,400	344,500	346,700	348,900	351,100	353,400	355,600	357,800	360,000	362,200	364,400	366,600	62
63	335,300	337,500	339,700	341,800	344,000	346,200	348,400	350,600	352,800	355,000	357,200	359,400	361,600	363,900	366,100	368,400	370,600	372,800	375,100	377,300	379,600	63
64	347,000	349,200	351,400	353,700	355,900	358,100	360,300	362,600	364,800	367,000	369,200	371,500	373,800	376,100	378,300	380,600	382,900	385,200	387,500	389,700	392,000	64
65	358,200	360,400	362,700	364,900	367,200	369,400	371,700	373,900	376,200	378,400	380,700	383,000	385,300	387,600	390,000	392,300	394,600	396,900	399,200	401,500	403,800	65
66	368,700	371,000	373,300	375,600	377,900	380,200	382,500	384,800	387,100	389,400	391,700	394,000	396,400	398,700	401,100	403,400	405,800	408,100	410,400	412,800	415,100	66
67	379,000	381,400	383,700	386,000	388,400	390,700	393,000	395,400	397,700	400,000	402,400	404,700	407,100	409,500	411,900	414,200	416,600	419,000	421,400	423,700	426,100	67
68	393,600	395,900	398,300	400,700	403,000	405,400	407,800	410,200	412,500	414,900	417,300	419,700	422,100	424,500	426,900	429,400	431,800	434,200	436,600	439,000	441,400	68
69	408,000	410,400	412,900	415,300	417,700	420,100	422,500	424,900	427,400	429,800	432,200	434,700	437,100	439,600	442,000	444,500	447,000	449,400	451,900	454,300	456,800	69
70	422,500	425,000	427,400	429,900	432,300	434,800	437,300	439,700	442,200	444,600	447,100	449,600	452,100	454,600	457,100	459,600	462,100	464,600	467,200	469,700	472,200	70
71	437,100	439,600	442,100	444,600	447,100	449,600	452,100	454,600	457,100	459,600	462,100	464,600	467,200	469,700	472,300	474,800	477,300	479,900	482,400	485,000	487,500	71
72	451,500	454,100	456,600	459,200	461,700	464,300	466,800	469,400	471,900	474,400	477,000	479,600	482,200	484,800	487,400	490,000	492,600	495,200	497,800	500,400	503,000	72

HEADWATER 354 to 356

TAILWATER 338.01 to 341.00

Figure 92.—*Page from Kentucky Dam spillway discharge tables.*

In planning the spillways of the main-river dams, discharge capacity at both the maximum and minimum levels was investigated. It was required, of course, that the spillway be able to pass the regulated design flood flow at the maximum water level. Where possible, this level was set at the top of the spillway gates, but physical limitations of the natural river channel caused higher levels at Pickwick, Guntersville, Hales Bar, and Chickamauga. In addition to discharging the design flood at the maximum level, it was important that the main-river dams have a relatively high discharge capacity at the minimum level so that the storage space would not need to be filled early in the flood period. With the exception of Wheeler, which has 60 gates 15 feet high, this was accomplished with a moderate number of gates 32 feet, 40 feet, or 50 feet high. This low level capacity was particularly important at Kentucky Dam, where the relative timing of floods on the Tennessee and Ohio Rivers requires that Kentucky Reservoir storage be preserved for long periods during which Tennessee flow may be high. Consequently, 24 spillway gates 50 feet deep and 40 feet wide were provided here, giving (without tailwater submergence) a discharge capacity of 274,000 cubic feet per second at the minimum elevation of 346. The number of 40-foot-wide gates could have been reduced to 21 if the spillway design flood of 960,000 cubic feet per second had been the only consideration.

Sluiceways through the main-river dams were not required because sufficient discharge could be passed over the spillway to maintain the minimum levels during moderately high inflows. At time of high flows, sluiceways would not add greatly to the discharge capacity.

Spillways of the tributary storage dams were designed to pass the design flood as regulated by storage in the immediate reservoir or by any upstream storage. At Cherokee and Douglas, however, no reduction in the design flow for upstream storage regulation was made, because these projects were built during the war when it was planned that during that emergency they would be operated principally for power generation, with a consequent possible high reservoir level when and if the design flood were to occur.

Sluiceways were provided at all tributary storage dams to assist in drawing down the reservoirs to January 1 levels when turbine operation would be insufficient to do this, and to normal levels after a flood operation.

Since the completion of these dams, elaborate discharge tables have been prepared for most of the projects, and others are in preparation. These tables give discharge for various gate settings and headwater and tailwater levels to aid the operating force in setting the gates for any required discharge. The tables also provide for the use of outlets in such a manner as to minimize troublesome navigation currents and bank or bottom erosion.

Figures 92 and 93 are reproductions of three pages from the 336-page loose-leaf spiral-bound volume, *Kentucky Dam Spillway Discharge Tables.* Figure 92 shows one of the 332-pages of similar tables in the book, and figure 93 shows a diagram of the spillway gates and a key to the gate arrangements at the top with the gate arrangement tabulation at the bottom.

FUNCTIONS OF VARIOUS RESERVOIRS

The reservoirs in the TVA system fall into two general groups—main river and tributary—which are greatly different as to method of operation, relative flood storage capacity, and effect at critical points.

Tributary reservoirs

The ten flood storage reservoirs on the tributaries have a relatively large capacity with respect to flood volumes and, therefore, will be operated to store all or almost all the flood inflow. Various assumptions of tributary reservoir operation were made in the course of the planning studies, but it was found generally that the greatest benefit at Chattanooga, the critical point of flooding in the upper Basin, would result from storing the entire inflow from the beginning of the flood until such time as releases would not increase the peak flow at Chattanooga. Consideration of still other factors, however, such as the storage space available in each reservoir at the start of a flood; the capacity of the outlet works at various levels; flood control requirements below each tributary dam; and the generation of power; all indicated that there should be some discharge from these reservoirs from the beginning of the flood. Various methods of operation are described in subsequent pages and their results are compared.

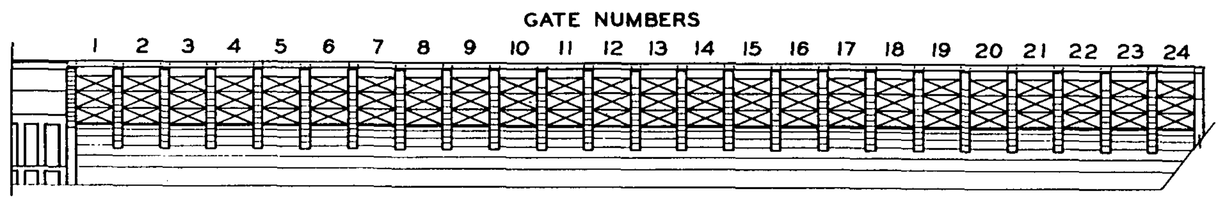
Main-river reservoirs

The second group of reservoirs, those eight on the Tennessee River which have a flood storage reservation, have a relatively much smaller flood storage capacity than the tributary reservoirs. For example, the average March 15 storage reservation on the tributaries is equivalent to 6.43 inches over their drainage areas, compared with only 1.79 inches in the main-river reservoirs, excluding Kentucky. Consequently, main-river reservoir operation during floods must be different from tributary reservoir operation. Instead of storing a large part of the inflow from the beginning of a flood, the best operation would be to hold the limited storage space empty by releasing the inflow (except for that which unavoidably goes into slope or profile storage), and then when the flood crest from the area downstream from the tributary dams arrives, the empty storage space would be filled. The storage space would be emptied as quickly as practicable after the flood to prepare these main-river reservoirs for a following

KENTUCKY DAM

SPILLWAY DISCHARGE

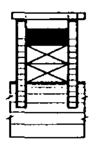
LOCATION OF SPILLWAY GATES



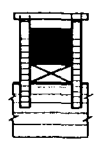
DOWNSTREAM ELEVATION

KEY TO GATE ARRANGEMENT TABLES

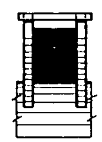
SYMBOLS REFER TO GATE OPENINGS AS USED IN THE GATE ARRANGEMENT TABLE, PAGE 4



U
UPPER LEAF REMOVED



M
UPPER AND MIDDLE LEAVES REMOVED



L
ALL LEAVES REMOVED

KENTUCKY DAM

SPILLWAY GATE ARRANGEMENTS

TENNESSEE VALLEY AUTHORITY

HYDRAULIC DATA BRANCH

GATE ARRANGEMENT	GATE NUMBER																							
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
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3	—	—	—	—	—	—	—	—	—	—	U	—	—	—	—	—	U	—	—	—	—	U	—	—
4	—	—	—	—	U	—	—	—	—	—	U	—	—	—	—	—	U	—	—	—	—	U	—	—
5	—	U	—	—	U	—	—	—	—	—	U	—	—	—	—	—	U	—	—	—	—	U	—	—
6	—	U	—	—	U	—	U	—	—	—	U	—	—	—	—	—	U	—	—	—	—	U	—	—
7	—	U	—	—	U	—	U	—	U	—	U	—	—	—	—	—	U	—	—	—	—	U	—	—
8	—	U	—	—	U	—	U	—	U	—	U	—	U	—	—	—	U	—	—	—	—	U	—	—
9	—	U	—	—	U	—	U	—	U	—	U	—	U	—	U	—	U	—	—	—	—	U	—	—
10	—	U	—	—	U	—	U	—	U	—	U	—	U	—	U	—	U	—	U	—	—	U	—	—
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20	—	U	—	—	U	—	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U
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25	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	U	M	U	U
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35	U	M	U	U	M	U	M	U	M	U	M	U	M	U	M	U	M	U	M	U	M	M	U	U
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GATE ARRANGEMENT	GATE NUMBER																							
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70	M	M	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L
71	M	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L
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JUNE 1960

GATE OPENING LEGEND

Gate openings are shown in the vertical columns under each Gate Number as follows:

U - UPPER LEAF OUT
M - MIDDLE LEAF OUT
L - LOWER LEAF OUT

FIGURE 93.—*Spillway gates diagram, key to gate arrangements, and gate arrangement tabulation—Kentucky Dam.*

flood. Figure 114, page 179, shows the actual 1946 operation of Chickamauga Reservoir.

Storage reservations for flood control

The summary of the system projects on the first page of this chapter, starting with the second paragraph, mentions the 18 multiple-purpose projects where capacity for storing flood water is assured. This capacity, in varying amounts in the 18 reservoirs, totals more than 11.8 million acre-feet on January 1 each year, about 10.4 million acre-feet on March 15, and 2.5 million acre-feet during the summer period. Some important flood control features of these 18 reservoirs are contained in table 26, page 147, and figure 94, which is a composite multiple-purpose guide curve for the system, shows graphically the total system flood storage reservations given in the table.

Because property along the main stem of the Tennessee River and all its tributaries has been subject to severe flood damage in the past, control of or protection from floods was sorely needed throughout the Basin. Likewise, the occurrence of disastrous floods in the lower Ohio and Mississippi Rivers, particularly at and below Cairo, makes reduction in the contribution of the Tennessee River to flood crests on these streams highly desirable.

Table 31 gives the storage space available in the TVA reservoir system above the critical points: Elizabethton, Kingsport, Knoxville, and Chattanooga in the Tennessee River Basin; Paducah on the lower Ohio River; and Cairo at the confluence of the Ohio and Mississippi Rivers.

TABLE 31.—*Available level storage space in TVA reservoir system upstream from critical points on pertinent dates.*

Critical point	River	Acre-feet[1] of level storage space available: January 1	March 15	Summer period
Elizabethton	Watauga	256,000	155,000	109,000
Kingsport	South Fork Holston	636,000	415,000	215,000
Knoxville	Tennessee	3,093,000	2,242,000	338,000
Chattanooga	Tennessee	6,822,000	5,421,000	1,205,000
Paducah	Ohio	11,814,000	10,413,000	2,485,000
Cairo	Mississippi	11,814,000	10,413,000	2,485,000

[1]. Rounded off to nearest 1000.

Relative value of reservoirs

The amount of flood regulation accomplished at a critical point by a reservoir or group of reservoirs depends on (1) their location with respect to the flood-producing storm; (2) the size of the drainage area controlled by them with respect to the total area above the place to be protected; and (3) the storage available at the beginning of a flood.

The three most upstream projects in eastern Tennessee are Boone, Watauga, and South Holston. Watauga will be operated principally for flood regulation at Elizabethton and Kingsport, and Boone and South Holston for Kingsport. A high degree of protection is provided for these cities by these three reservoirs in the winter flood season. Although intense storms and floods have occurred and will continue to occur in the summer in that area, a smaller

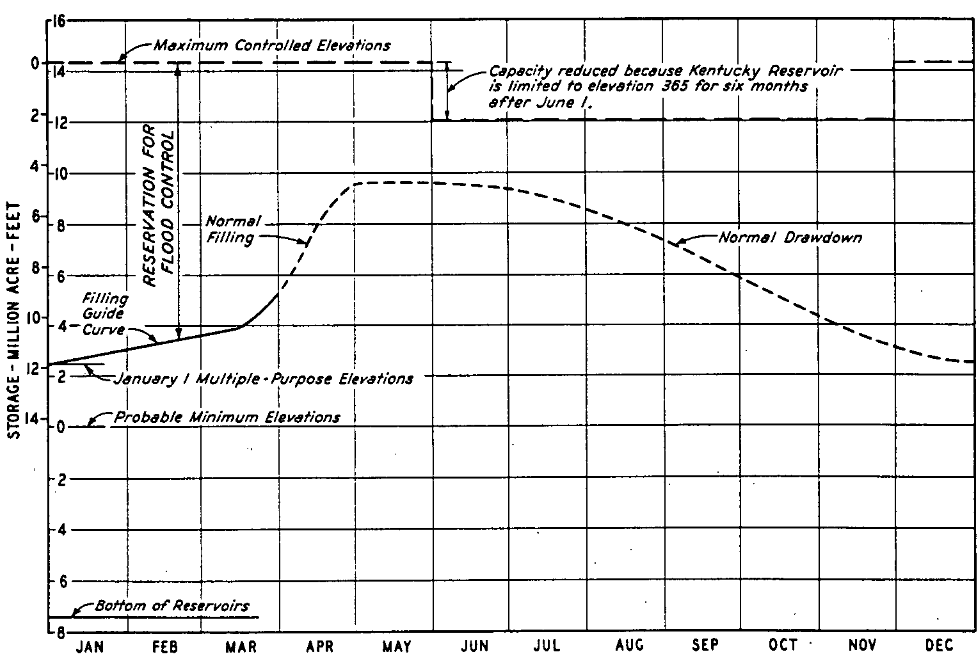


FIGURE 94.—*Composite multiple-purpose guide curve for the flood control system.*

storage reservation is provided because such floods have a smaller volume. The effect at Chattanooga and at Cairo of these three reservoirs will be small. The principal secondary benefit of Watauga and South Holston will be in providing a greater flexibility in the operation of Cherokee Reservoir.

Flood storage in the other seven tributary reservoirs and in the three Tennessee River reservoirs above Chattanooga is used primarily to reduce flood stages at Chattanooga. Substantial reductions accomplished at Chattanooga will be accompanied by similar reductions between the tributary dams and the Tennessee River and in the upper reaches of each main-river reservoir. Considering main-river and tributary reservoirs as two separate groups, the seven tributary reservoirs above Chattanooga will account for more than one-half the peak reduction at that point, with the three main-river reservoirs accounting for the remainder.

All reservoirs in the Basin will, of course, have some effect on stages in the Tennessee River. However, the normal available space in small reservoirs controlling only a limited area and having no reservation for flood control is not considered dependable for the regulation of floods.

The available storage in Guntersville, Wheeler, Wilson, and Pickwick Reservoirs is relatively limited, as shown in table 26, page 147. A favorable operation of these reservoirs, therefore, is to reduce the peak discharge as low as possible below each of the dams, using the available storage at rates consistent with runoff predictions and current stages on the lower Ohio and Mississippi Rivers. This operation will benefit agricultural land downstream from each dam which would be flooded under natural conditions. Releases after the flood will be governed by flood conditions on the Ohio and Mississippi Rivers and will be coordinated with the operation of Kentucky Reservoir.

Kentucky Reservoir affects stages on 22 miles of the Tennessee River and on the lower Ohio and Mississippi Rivers. Because the Ohio River is the principal contributor to Mississippi River floods and the Tennessee River is the largest tributary of the Ohio River, Kentucky Reservoir is at a highly favorable location for reducing flood stages at Cairo. Being located only 69 miles from Cairo — between one and two days of water travel—it can be closely regulated for requirements at that point and will influence the flow from nearly the entire Tennessee River Basin.

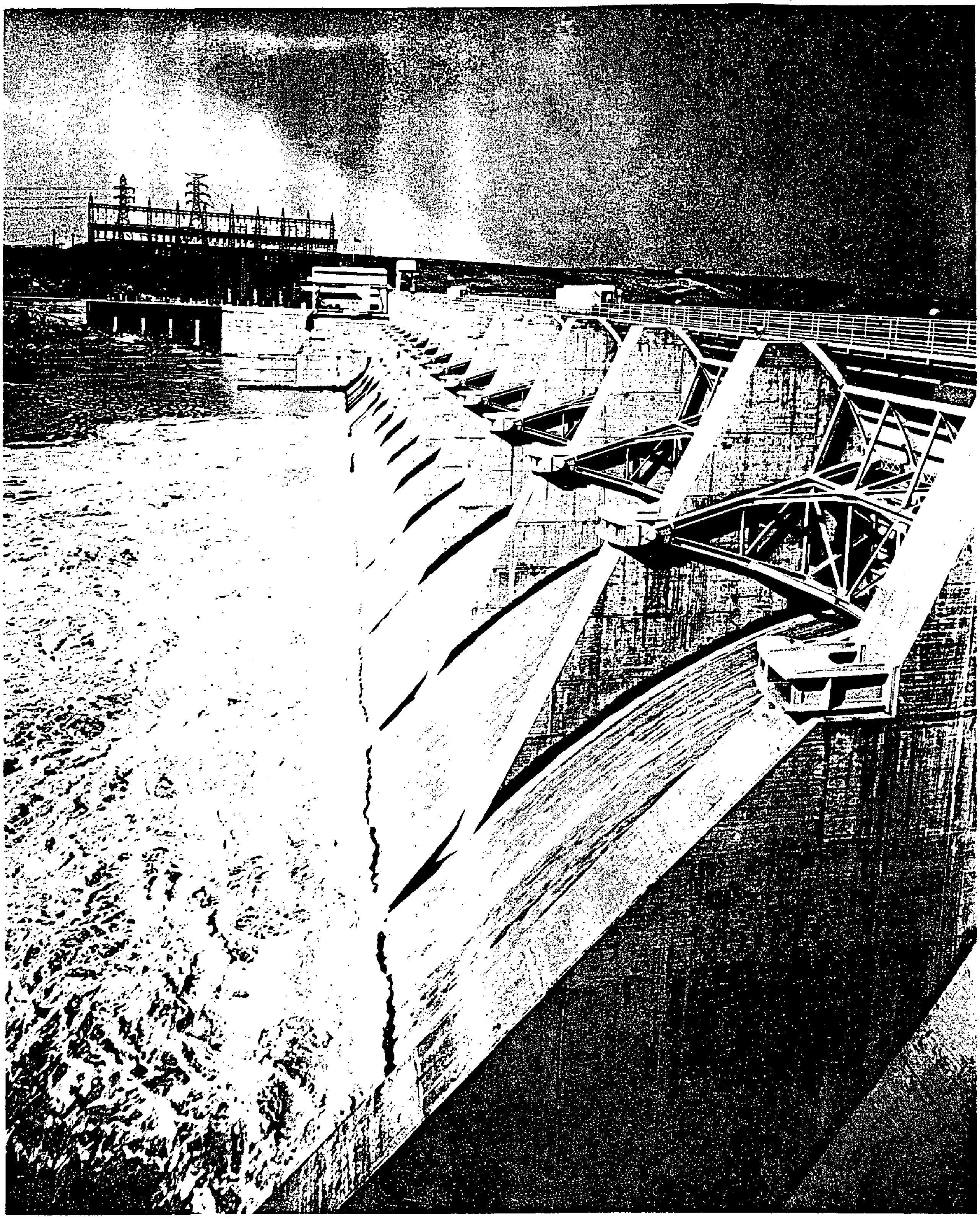


FIGURE 95.—*Fort Loudoun Dam spilling water in excess of turbine requirements to bring reservoir to flood season level.*

CHAPTER 9

ACTUAL RESERVOIR OPERATION FOR FLOOD CONTROL

The actual operation of the TVA reservoir system for flood control is covered in this chapter which discusses the application of operating principles, describes the many factors entering into the mechanics of TVA reservoir operation, includes an itemization of the work involved, and tells of the problems—both temporary and continuing—affecting flood control operations.

APPLICATION OF OPERATING PRINCIPLES

The general methods described in chapter 7 are followed in operating the system reservoirs for flood control.

Seasonal operation

The tributary multiple-purpose reservoirs are lowered to their minimum flood season levels by January 1. They are allowed to fill slowly until March 15, as the probability of the occurrence of large floods decreases, and then more rapidly until April 1. If a flood occurs during this period, some or all the storage space above these levels may be used temporarily in regulating flows, after which water would be released to return the reservoirs to normal filling levels. After April 1, if water is available, they may be filled to normal full levels, reserving a small amount of space for the regulation of summer floods. During the summer and fall this stored water is released, usually through the turbines, to return the reservoirs gradually to the January 1 flood season levels. If turbine releases are not sufficient, gate discharge is made to complete the drawdown. Figure 96, the guide curve for Norris Reservoir on the Clinch River, is typical of the operation of the tributary multiple-purpose reservoirs.

The main-river reservoirs are also lowered to their minimum flood season levels by January 1. These levels are held until the end of the flood season, except during flood control operations when some or all the flood storage space may be used temporarily in regulating flows, after which the water is released as rapidly as is consistent with flood control objectives. Between March 15 and May 1 the reservoirs are filled gradually to summer levels. If water is available, a temporary surcharge above these levels is made for the purpose of stranding drift as an important part of malaria control operations. The reservoirs are then held near summer levels for a few months. Seasonal recessions are then started to aid malaria control, and are continued in order to lower the reservoirs to flood season levels by the end of the year. As a further aid to malaria control, weekly fluctuations of water levels to strand and destroy larvae of the malaria-bearing mosquito are made in addition to the general recessions. Figure 97, the guide curve for Chickamauga Reservoir, is typical of the operation of the main-river reservoirs.

Tributary reservoir operation

The adopted plan of operation for flood control provides for using the multiple-purpose tributary reservoirs, all of which are above Chattanooga, for temporarily storing as much of the storm runoff as possible until the flood crest has passed Chattanooga. It would be desirable to store all the runoff in those reservoirs during the storm period, except a relatively small amount needed for turbine use. Early studies showed that this plan could not be followed in the great floods because of lack of sufficient capacity for storing all the runoff. The release necessary during such flood periods, however, would be small compared with the total amount of runoff entering the reservoirs.

Main-river reservoir operation

The three multiple-purpose mainstream reservoirs above Chattanooga supplement the tributary reservoirs in reducing flood stages at that city. The flood storage space in these three reservoirs is in the order of only 1 or 2 inches of runoff over the local drainage areas compared with about 5 to 10 inches in the tributary reservoirs. Consequently, it is possible to store only a small part of the runoff in these mainstream reservoirs during a flood period. The plan of operation contemplates, whenever practicable, a drawdown, or at least not a rise, of the headwater at each of the three mainstream dams

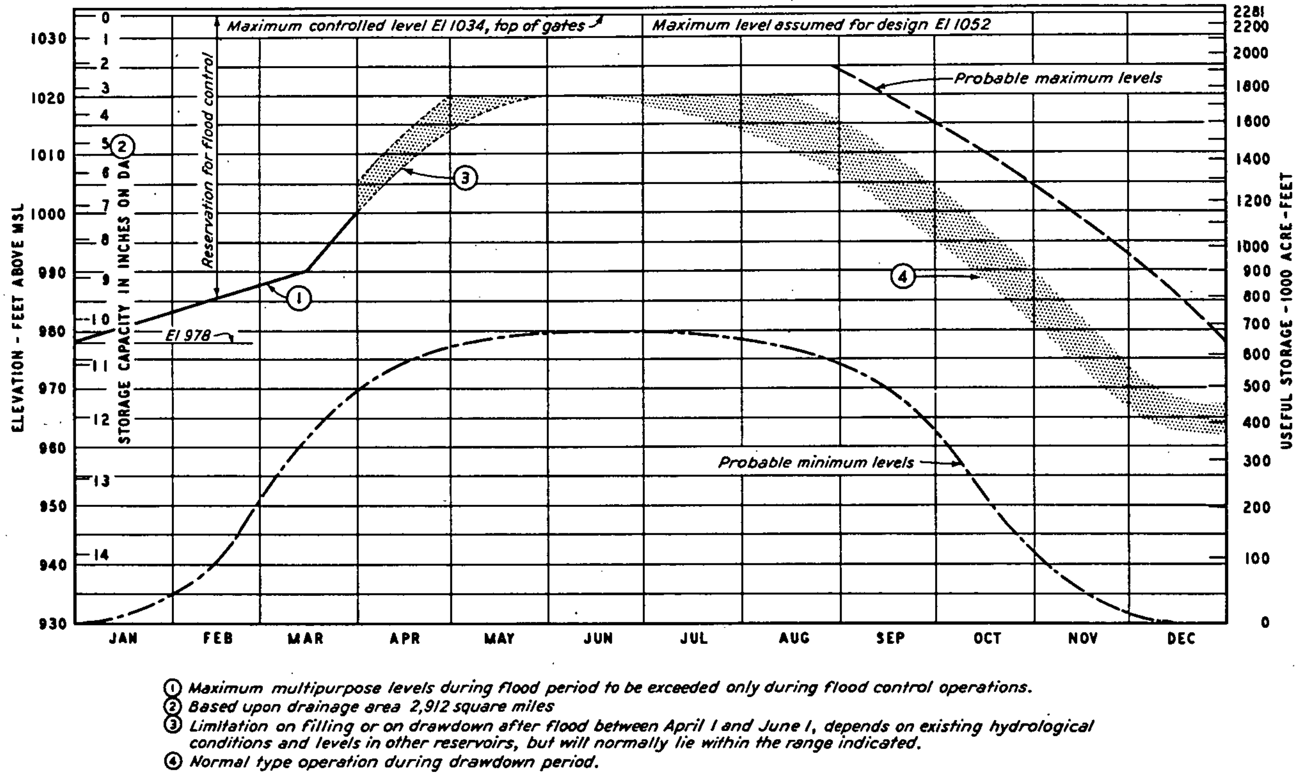


FIGURE 96.—*Multiple-purpose operating guide curve—Norris Reservoir.*

above Chattanooga during the early part of the flood as the flow increases. This operation tends to compensate for the loss in reservoir storage volume occasioned by slope storage as the flow increases, and so long as the outflow is made equal to the inflow most of the flat pool storage volume can be maintained for later use.

Storing for crest reduction—After this initial operation in the mainstream reservoirs has been completed and as the flood approaches its crest stage, any or all the remaining storage space in these three reservoirs may be used as considered advisable for a particular flood. With proper timing of the filling storage space it is feasible to reduce the flood crest at Chattanooga to a much greater extent than the reduction afforded by the tributary reservoirs alone.

Release after flood crest—After the passage of the crest at Chattanooga, the release of the water stored in these three mainstream reservoirs, along with that stored in the multiple-purpose tributary reservoirs farther upstream, is made as rapidly as is safe. By this means the water level in the three mainstream reservoirs is returned to minimum flood season level promptly, and the reservoirs are ready again for any subsequent flood. Likewise, the release from tributary reservoirs after the passage of the flood crest at Chattanooga restores the required flood storage space in them for future use, but some of the floodwater stored in these reservoirs is retained for later use during the dry season.

Early operation of Norris Reservoir

In general, the operation of TVA reservoirs for flood control—especially for Chattanooga—began early in March 1936 with the closure of Norris Dam, the first multiple-purpose project completed above that city. At that time only two other multiple-purpose projects had been completed, Hales Bar in 1913 and Wilson in 1925, but when these projects were originally built no provision was made for flood control. Both are on the main Tennessee River downstream from Chattanooga. During March and April 1936 the water withheld in the early stages of the initial filling of Norris Reservoir reduced three flood crests at Chattanooga. Two of these floods would have reached the fairly high stages of 38.8 and 41.3 feet but were reduced 3.4 and 4.2 feet to crest heights of 35.4 and 37.1 feet, respectively. The third flood crest barely exceeded the flood stage of 30 feet and was reduced about ½ foot.

Again, in January and February 1937, three flood crests of somewhat lower height, but of long duration and comparatively large volume, occurred at Chattanooga. These crests were reduced some 2 to 3 feet by the operation of Norris Reservoir.

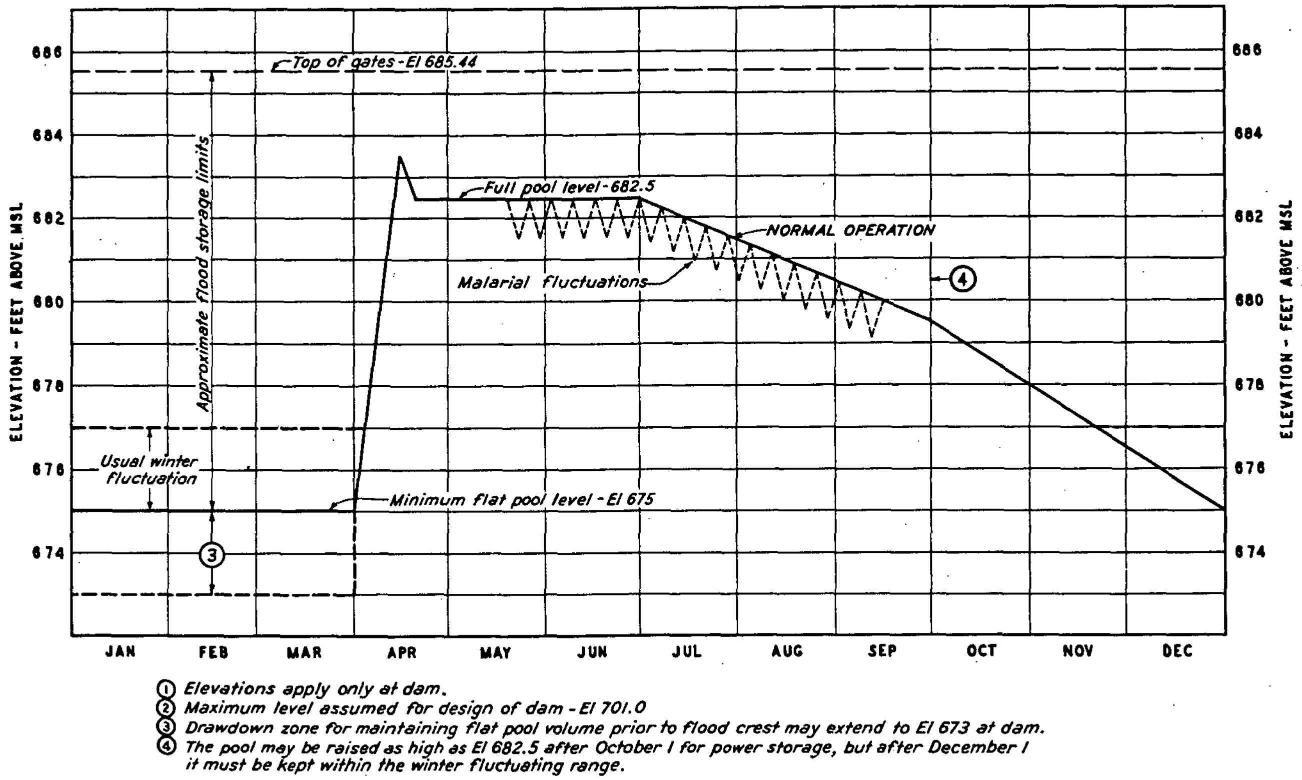


FIGURE 97.—*Multiple-purpose operating guide curve—Chickamauga Reservoir.*

Early studies and operations of only one multiple-purpose tributary reservoir (Norris) for flood control demonstrated that under certain flood conditions relatively large releases would have to be made from tributary reservoirs. In February 1937 water was released from Norris Reservoir (fig. 98) at a maximum rate of approximately 40,000 cubic feet per second for a short time, after the last of a series of crests at Chattanooga, to restore flood storage space in the reservoir for subsequent use. This operation of the reservoir was abnormal, not being required to the same extent since that time and not expected in the future or, at the most, rarely. Throughout that long and almost continuous flood period of nearly two months, Norris Reservoir also was used to reduce river stages to protect construction work in progress at Chickamauga, Guntersville, and Pickwick Landing Dams. This use resulted in filling the reservoir considerably in excess of normal requirements for flood control.

Operation of reservoirs for Chattanooga

The principal tributary reservoirs are within only about one to three days' time of floodwater travel from Chattanooga, and they regulate the flow from 13,420 square miles, nearly 63 percent of the drainage area above Chattanooga. This tributary regulation makes it practicable for the mainstream reservoirs, with their limited amount of storage, to attain beneficial regulation of the flow from the area between the tributary reservoirs and Chattanooga. Since Chattanooga was the critical location of flood hazard, the sites of tributary storage projects were selected so as to provide the optimum flood regulation at this key city.

In the 23-year period since closure of Norris Dam in 1936, there have been 33 floods that would have exceeded flood stage of 30 feet at Chattanooga had it not been for the reservoir system. The greatest flood at Chattanooga since the completion of the existing reservoir system occurred in January-February 1957. This flood would have reached a stage of approximately 54 feet, the second-highest flood of record, had it not been for the reservoir system. The actual peak stage with regulation was 32.2 feet, about 22 feet lower than the computed natural peak. This substantial reduction was possible because the storm, which extended over a continuous period from January 24 to February 5, was relatively heavy over the drainage areas contributing to the tributary storage reservoirs. The reduction was also aided slightly because those reservoirs, as the result of an extended dry fall and early winter with heavy power demands, were at levels lower than those required for flood operation. Figure 99 shows the hydrographs—with and without reservoirs—at Chattanooga during the January-February 1957 flood period, and

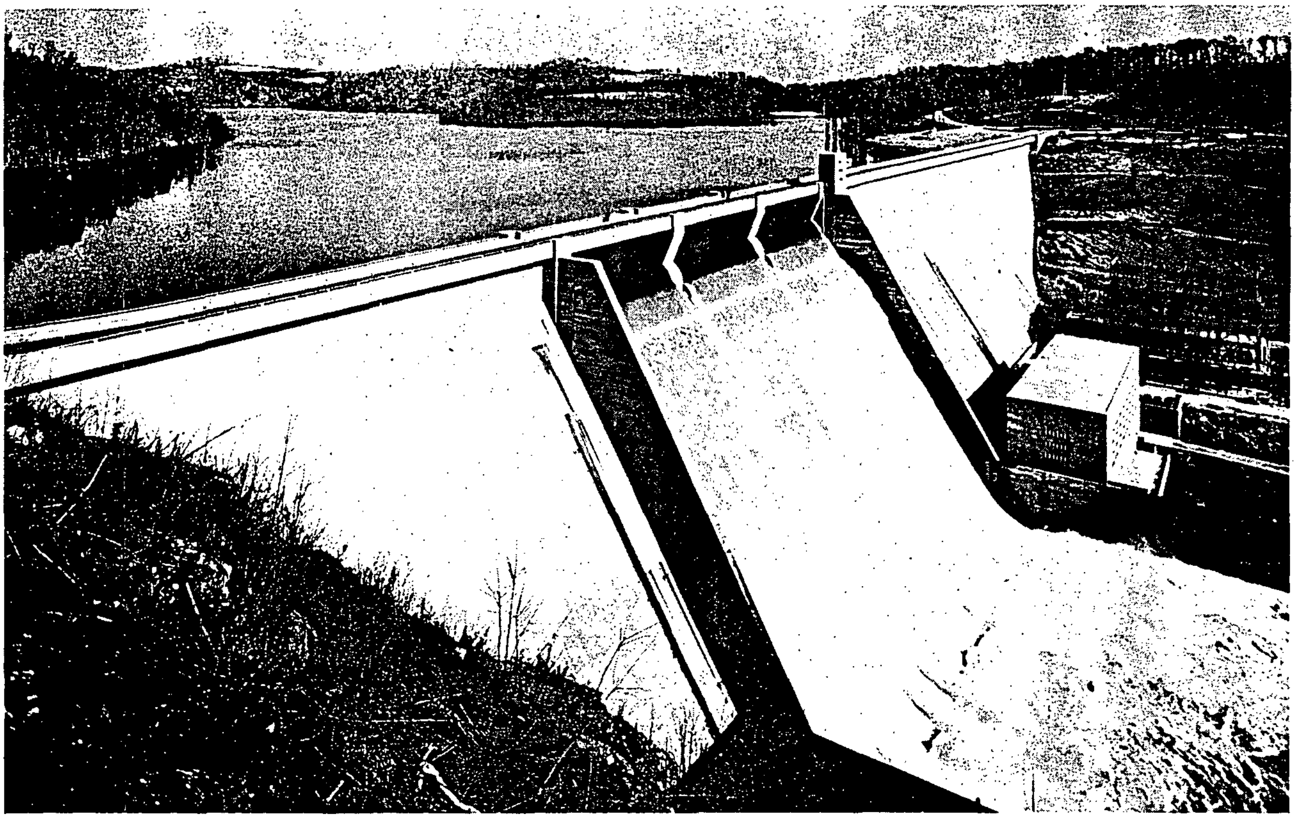


FIGURE 98.—*Norris Dam releasing stored flood water—February 1937.*

figure 100 shows Chickamauga Dam on February 12, 1957, releasing the flood waters at a rate of 146,600 cubic feet per second. The area actually flooded in Chattanooga compared with the area that would have been flooded had there been no system regulation is shown by figure 101.

Tributary storage projects impounded practically all the inflow with a minimum release to supply the basic power requirements. In the case of Norris Reservoir, the inflow reached a rate of 86,000 cubic feet per second early in the flood, while at the same time the discharge was held to about 7,000 cubic feet per second. At Douglas Reservoir the inflow reached a rate of 110,000 cubic feet per second, with a discharge at that time of 15,000 cubic feet per second. At other tributary reservoirs the same pattern was followed. After danger at Chattanooga had passed, the discharges at the tributary reservoirs were increased to return them to their normal seasonal levels. For example, during this period discharges were as much as 26,000 cubic feet per second at Norris and 33,000 cubic feet per second at Douglas.

Operation for lower basin

The operation of the reservoir system for Chattanooga will reduce flood flows into the reach just below Chattanooga, which is beneficial for the lower portion of the basin.

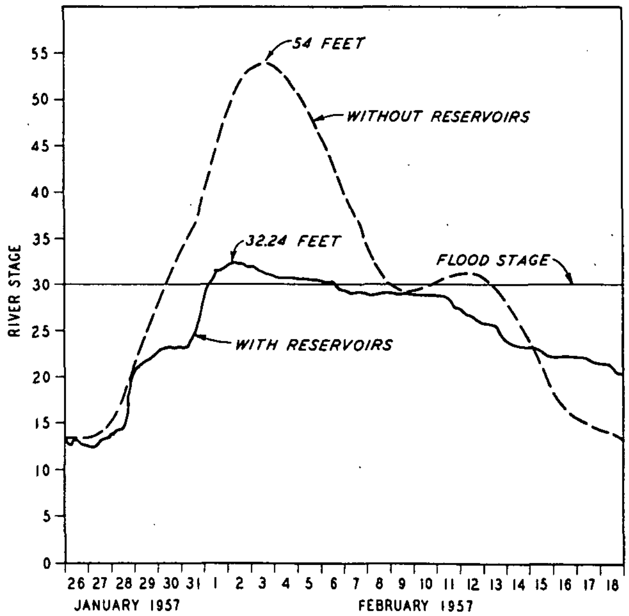


FIGURE 99.—*Hydrographs of the January-February 1957 flood at Chattanooga.*

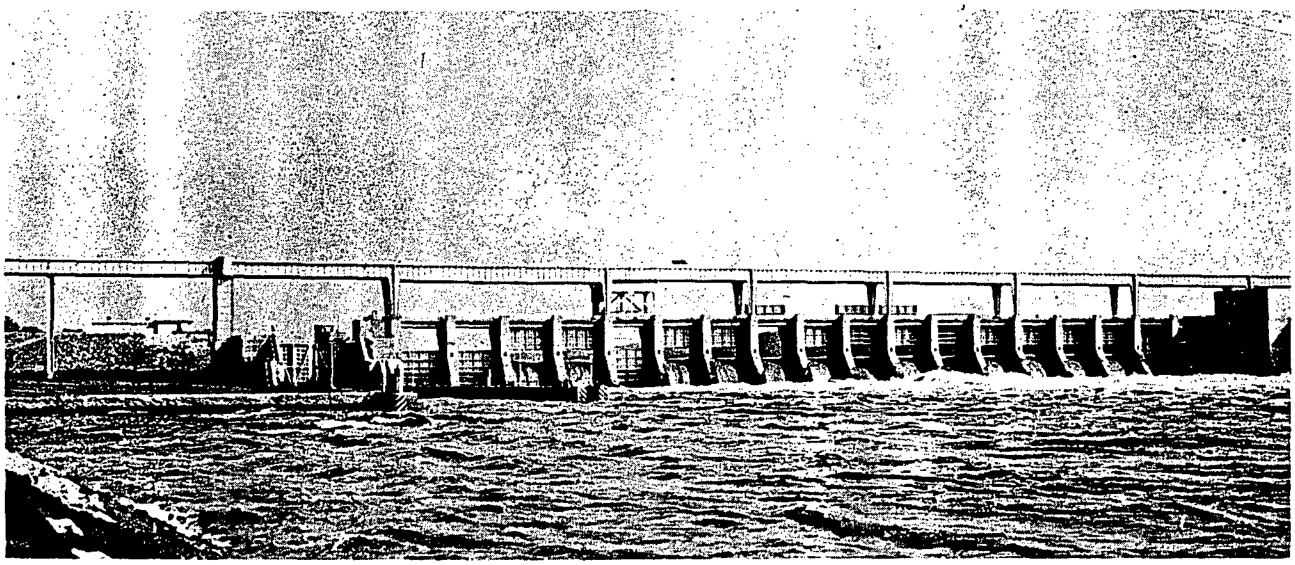


FIGURE 100.—*Chickamauga Dam releasing flood waters at rate of 146,000 cubic feet per second on February 12, 1957.*

With the exception of Wilson and Kentucky, the mainstream reservoirs below Chattanooga are planned to be operated in a manner similar to that of the mainstream reservoirs above Chattanooga. Provision is made for an initial drawdown in the headwater at Guntersville and Wheeler Dams in advance of the flood peak to preserve the limited flood storage space for use during the critical crest period. Then water will be stored during the passage of the flood peak to reduce the peak in the river reach below each dam. Finally, after the flood crest advances downstream, the water will be drawn out of these reservoirs as rapidly as practicable, thereby restoring flood storage space for future use. Although the flood storage space in Wilson Reservoir is small, it is important for regulating flood peaks at Florence, Alabama. The general plan for its operation and for Pickwick Reservoir is the same as in the other Tennessee River reservoirs, except that no drawdown is provided in advance of the flood crest. These exceptions are made since the upper guard sills of the locks are too high to permit more than nominal drawdown and, at the same time, maintain the depth required for 9-foot navigation.

Operation of Kentucky Reservoir for Ohio and Mississippi Rivers[1]

Kentucky Reservoir, with 4,000,000 acre-feet of flood storage capacity, is by far the largest and most important in the entire reservoir system for regulating releases from the Tennessee River Basin into the lower Ohio River and thence into the Mississippi River. It is strategically located, within one or two days water travel from Cairo, and thus has great value for re-regulating the flows from the entire TVA reservoir system.

The plan of its operation is somewhat different from that of the other mainstream reservoirs. Extensive studies were made in advance of its completion to determine the best method. Floods of record in the Tennessee River which affected floods in the lower Ohio and Mississippi Rivers were studied under various operating rules for the purpose of devising a practical plan of operation for reducing the flood crests on those rivers and, at the same time, keeping satisfactory water levels within the reservoir. The plan finally adopted provided during the flood season for a permissible drawdown of the headwater at Kentucky Dam of 8 feet (from elevation 354 to elevation 346) at a rate of about 1 foot per day in advance of the arrival of flood crests at Paducah and Cairo. This total drawdown is within the limitations of the design of the lock and other structures. The drawdown may occur even at the time of the Tennessee flood crest. The objective is the maintenance of constant storage volume in the reservoir, as flows increase, for impounding water in the reservoir during the Cairo flood crest.

The combined Ohio River and upper Mississippi River flows determine the crest of the flood in the Mississippi River at and below Cairo, and regulation of releases at Kentucky Dam are timed to effect the desired reduction in the crest at Cairo. After the Cairo flood crest has passed down the Mississippi, the floodwater stored in the reservoir is discharged as promptly as downstream conditions permit, thereby restoring the reservoir to normal flood season level. Thus, in this reservoir the plan of operation is to discharge the Tennessee River flood crest usually in advance of the Ohio and Mississippi flood peak instead of after that peak.

1. For participation of Corps of Engineers in this operation see "Effect of Section 7, Flood Control Act of December 22, 1944," on page 131 of this report.

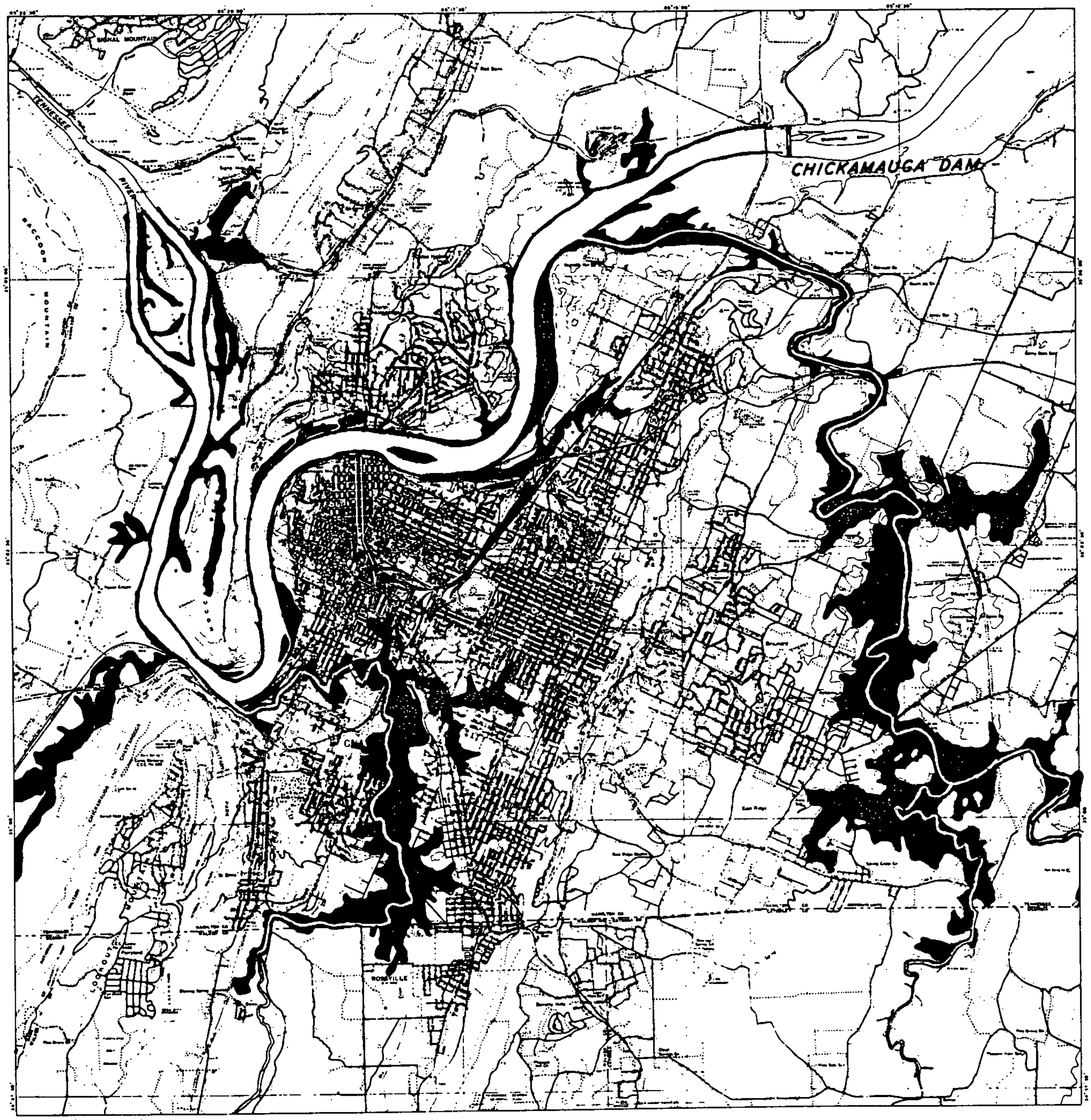
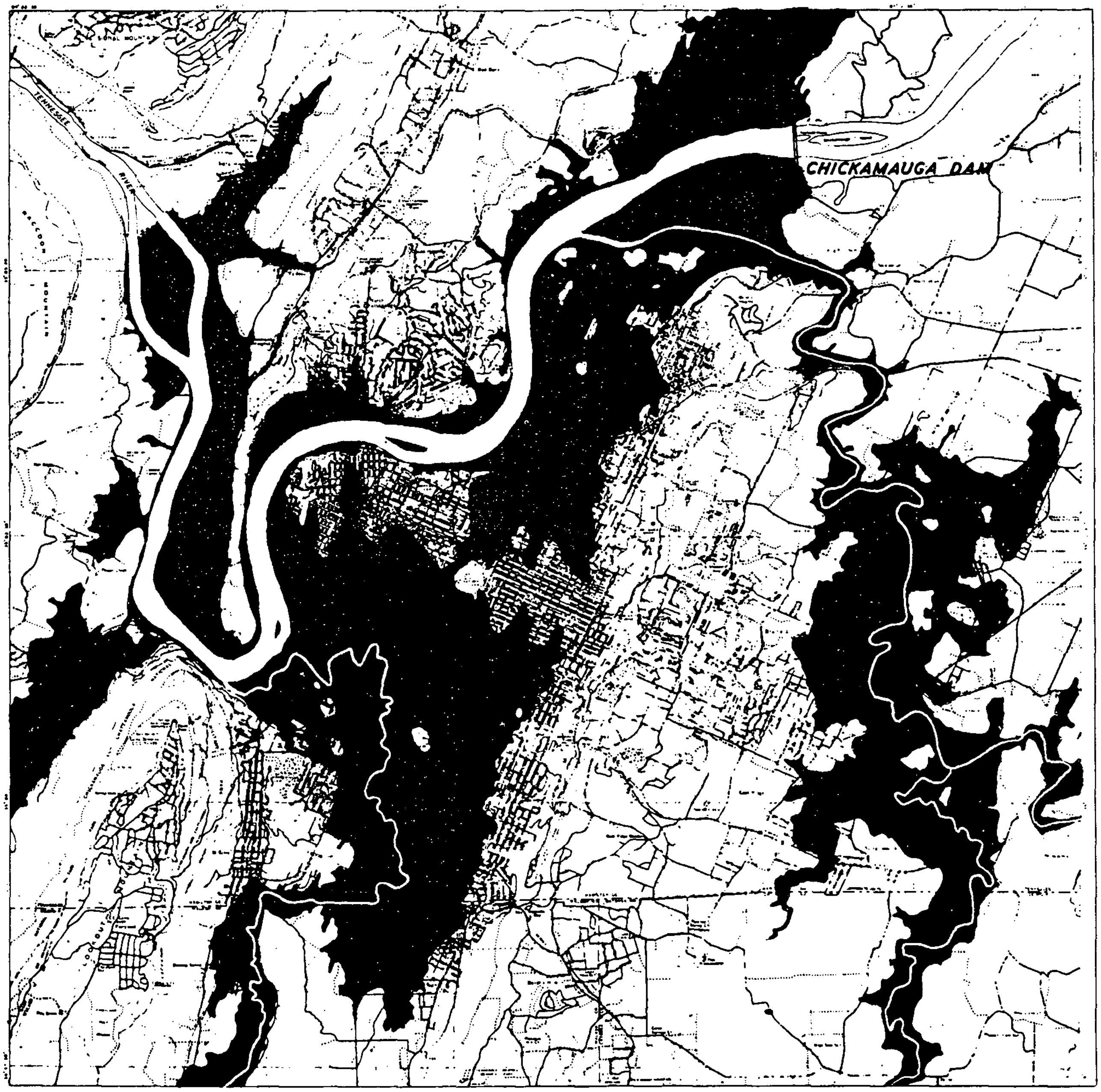


FIGURE 101.—*Maps showing Chattanooga as actually flooded (left) in 1957 com*



area that would have been flooded (right) had there been no regulation.

To accomplish this operation successfully, it is necessary to have predictions of expected flows and stages on the Ohio and Mississippi Rivers. During flood periods, stage forecasts are available from the U. S. Weather Bureau and discharge and stage forecasts from the Corps of Engineers.

Operation of the reservoirs above Chattanooga may, of course, affect stages on the lower river and also the operation of the lower-river reservoirs. Since the upper reservoirs are primarily for the benefit of Chattanooga and other points in the upper Basin storing of floodwater proceeds with crest reductions at those points as the major objective. The effect of such storage of floodwater on river stages below Chattanooga is beneficial, reducing the crest stage and duration of high flows. Operation of the upper Basin reservoirs after flood danger is over in that section takes into account flood conditions on the lower Tennessee, Ohio, and Mississippi Rivers; that is, reservoir drawdown is so regulated that it does not increase the burden on the Guntersville, Pickwick, Wheeler, and Kentucky Reservoirs.

Studies—supported later by actual operation—indicated that the TVA reservoir system was capable of reducing the crests of major floods on the Mississippi River at Cairo between 2 and 3 feet. Such reduction, of course, is achieved only when a flood in the Tennessee River is concurrent with the flood in the Mississippi River. If the Tennessee River should

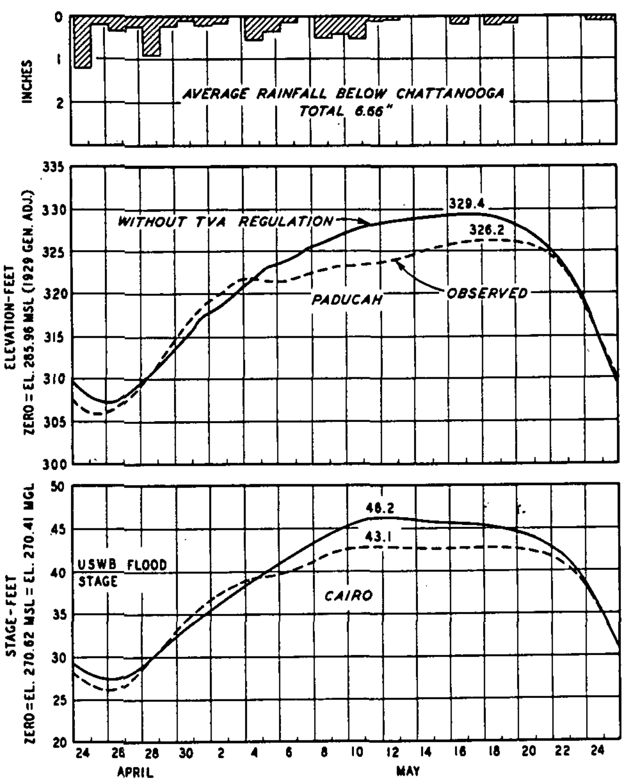


FIGURE 102.—*Hydrographs of the April-May 1958 flood at Paducah and Cairo.*

not be in flood simultaneously with the Mississippi River, or if the Tennessee River flood should be relatively small, the reduction which could be made in the Mississippi flood crest by regulation of the Tennessee River contribution would be proportionately less. Figure 102 shows the hydrographs—with and without regulation—at Paducah and Cairo during the April-May 1958 flood period. Although the Cairo crest was not unusually high compared with previous May floods, the reduction of 3.1 feet creditable to the TVA system was the maximum yet achieved at that city.

MECHANICS OF TVA RESERVOIR OPERATION

Day-to-day operation of the reservoir system requires frequent and detailed advance scheduling of discharges from each reservoir because none of the outlet works are automatic.

Responsibility—The responsibility for determining and scheduling reservoir operations is delegated to the Division of Water Control Planning through the Office of the Chief Engineer in Knoxville. The physical operation of the outlet works at the various dams is the responsibility of the Division of Power System Operations in the Office of Power in Chattanooga. Hence, instructions for the operation of the system originate with the Division of Water Control Planning and are issued to the Division of Power System Operations for execution. Instructions of a broad nature or those involving special operations are issued formally in Water Control Memoranda. Instructions on day-to-day scheduling and operation are transmitted by telephone and by facsimile.

Water control memoranda—During the 24 years (1936-1959) of experience in operation of the system, methods and procedures have been developed gradually and have been established through the issuance of Water Control Memoranda. In the earlier years of operations, as many as 154 memoranda were issued in one year (1939) giving detailed instructions on the operation in progress. In recent years, as the methods of operation have become well known and understood, the number issued has decreased to only a few anually, and normal instructions have been transmitted directly by telephone and by facsimile. Figure 103 is a copy of one of the early memoranda of instructions (Water Control Memorandum No. 783), issued on January 21, 1947, during the flood which occurred at that time.

Activities involved in operation

Operation of this large system of multiple-purpose reservoirs for flood control involves a large amount of detailed work, much of it of a technical or scientific nature. Most of this work must be done under the pressure of time limitations and sometimes over extended periods of days. It includes:

STANDARD FORM NO. 64

TENNESSEE VALLEY AUTHORITY

Office Memorandum • UNITED STATES GOVERNMENT

TO : Mr. C. L. Karr, Director, Power Operations Department, Chattanooga DATE: January 21, 1947

FROM : James S. Bowman, Chief Water Control Planning Engineer, Knoxville

SUBJECT: WATER CONTROL MEMORANDUM #783

This confirms telephone conversations with your office on Saturday and Sunday, January 18 and 19, as follows:

> In view of the rainfall which has occurred during the current week and of the predicted heavy rainfall over the week-end, it is necessary to continue the regulation of discharges at all mainstream and tributary dams in such manner as to best control the flood in the Tennessee River and its contribution to the lower Ohio and Mississippi Rivers in the event floods in those rivers reach dangerous proportions.
>
> To accomplish the above, discharges at all mainstream dams should be gradually increased today and thereafter, during the course of the flood, regulated in accordance with estimates furnished by this office from time to time.
>
> Discharges at Norris, Douglas, Cherokee, Fontana, and Hiwassee Reservoirs should be limited to turbine discharge.

Weather predictions yesterday and today (January 20 and 21) indicate that the storm which produced the current flood in the Tennessee River has passed and that the prospects are that there will be no heavy rainfall during the next 48 hours or more. Accordingly, discharges at mainstream dams should continue to be regulated as per estimates furnished by this office with a view to reducing the flood peak at Chattanooga, and all along the river as much as practicable. Also, discharges at Kentucky Dam will be made, as indicated by this office, so as to continue the drawdown of the headwater at Kentucky Dam with a view to maintaining constant volume of storage space in Kentucky Reservoir, so far as practicable, for use in regulating the contribution of the Tennessee River to the lower Ohio and Mississippi Rivers in case the present flood in those rivers should reach dangerous heights and require regulation.

As soon as the flood crest has passed Chattanooga and down the Tennessee River, water should be discharged from Norris, Douglas, Cherokee, Fontana, and Hiwassee Reservoirs, as rapidly as safe downstream from the dams, in amounts and at such times as may be designated by this office to draw these reservoirs down to flood season levels. Water should also be discharged from the mainstream reservoirs after the passage of the flood crest so as to return these reservoirs to flood season levels as soon as practicable.

Recommended ____________________________

Nicholls W. Bowden

NWB:CC
CC to Persons Listed in Water
Control Memorandum #775

FIGURE 103.—*Water control memorandum.*

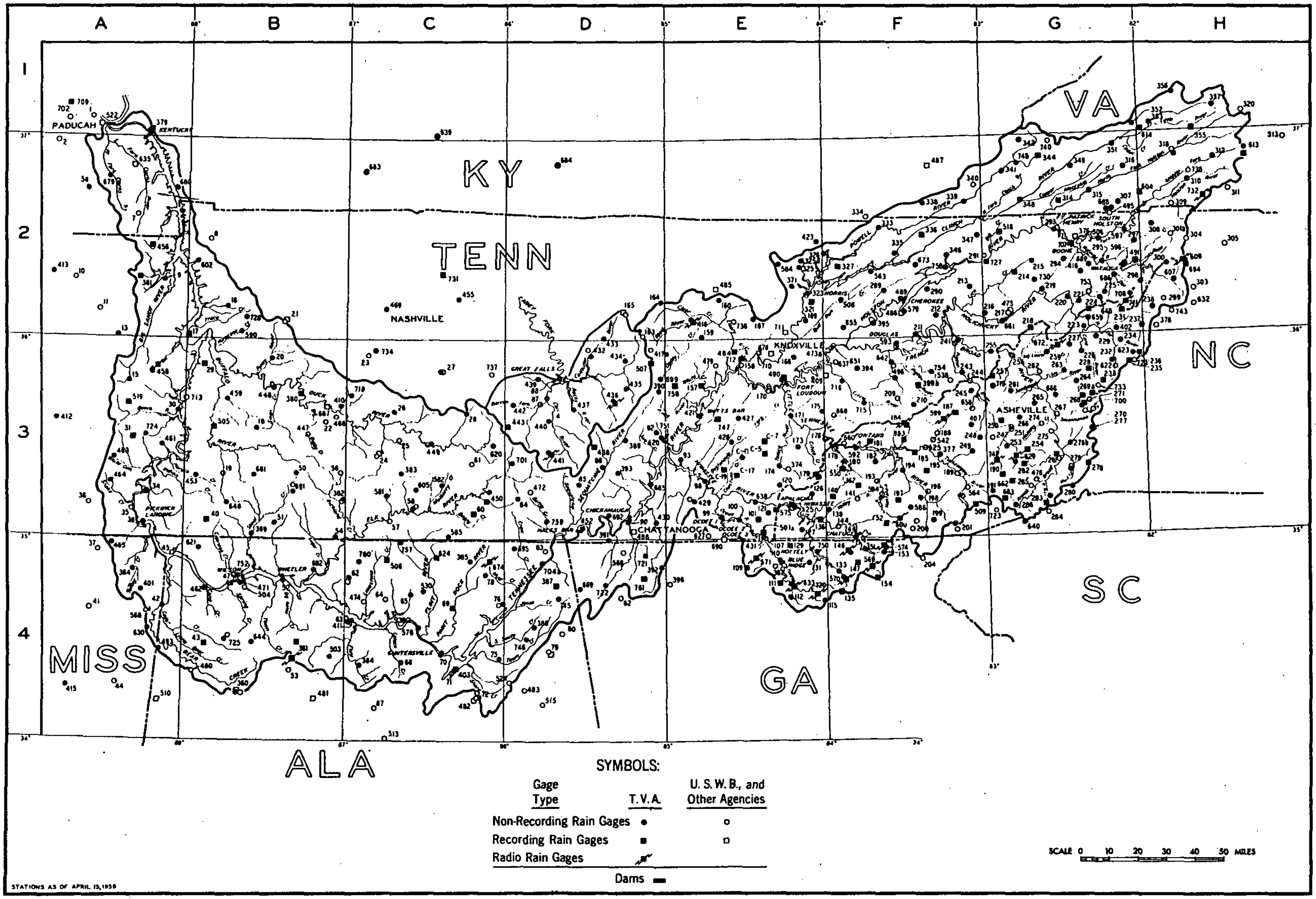


FIGURE 104.—*Rain gages in the Tennessee River Basin—April 1959.*

1. Observing, transmitting, receiving, and assembling data on rainfall and streamflow from the entire Basin.
2. Forecasting weather, by the U. S. Weather Bureau, including quantitative precipitation estimates.
3. Analyzing all the above data and converting them into current and anticipated runoff, taking into account both observed and predicted data.
4. Computing resulting runoff into each reservoir.
5. Receiving and analyzing observed and predicted flows and crests on Ohio and Mississippi Rivers furnished by Division offices of the Corps of Engineers located on those rivers, and by the U. S. Weather Bureau at Cairo.
6. Determining the proper operation of each reservoir to attain the desired objectives by routing the flows through the entire system and into the Ohio River.
7. Issuing instructions to the Division of Power System Operations concerning discharges to be made at all dams.
8. Furnishing the results of the above in terms of streamflow and stage to the U. S. Weather Bureau at Knoxville, Chattanooga, and Cairo and to the Corps of Engineers at Cincinnati.
9. Issuing a Daily River Bulletin in cooperation with the U. S. Weather Bureau for the information of the public.

Current hydrologic data—The forecasting of runoff for operation of the reservoir system requires reports from an extensive network of rainfall and streamflow stations. Each morning, reports are received in the River Control office in Knoxville from 190 rainfall and 41 streamflow stations. In addition, elevations and discharges are received from each reservoir. During critical periods, additional reports during the day may be required from most stations, and reservoir elevations and discharges are available through the power dispatching office in Chattanooga. Figures 104 and 105 show the locations of the rain gages and stream gages, respectively, in the Tennessee River Basin.

Of the 190 rainfall stations reporting daily, 54 are at power plants and substations and are observed by TVA personnel. The remainder are located as well as communication facilities permit to give an accurate measure of Basin rainfall. The majority are standard nonrecording gages, but 12 recording gages distributed throughout the Valley are used to measure rainfall intensity. Observers at these stations report by telephone. At certain locations in the Valley from which reports are desired, either communications or observers are not available, and 22 automatic radio rain gages are in use which broadcast amounts every two hours, thus also adding to the intensity network.

The 41 streamflow stations are located either on principal tributaries or on small areas which are used as indexes of flow. At 22 of these, observers abstract stages from the recorder charts and report by telephone. The remaining 19 are automatic radio gages which transmit stages at two-hour intervals. Elevations are received from 10 additional stations on the main river for which daily elevation forecasts are made. Hourly discharges from 33 dams in the system complete the streamflow picture.

Data collection—Ten TVA field offices covering the Valley serve as the collecting centers for the rainfall and streamflow data for their part of the Basin. Observers report to these offices early each morning. Automatic radio receivers in these offices record the broadcasts from the radio rainfall and streamflow gages in their area. At a fixed time each morning the telephone company completes a reserved call between the area office and the River Control office in Knoxville. On this call the area engineer transmits all the data for his area in a period of 5 to 10 minutes. Three aides in the Knoxville office receive the telephone calls and record the information on data forms. Complete information from this part of the data collecting system is received in a 35-minute period by 8:40 a.m.

The dispatching office of the Division of Power System Operations in Chattanooga collects hourly data on elevations and discharge at each reservoir in the system and twice daily observations of rainfall at 54 dams and substations. Each morning this information is transmitted to Knoxville by facsimile. Data to midnight of the preceding day are available in Knoxville by 7:00 a.m., and data for the first 6 hours of the current day are available by 8:30 a.m. During critical periods, current information on the system is obtained by telephone or by facsimile from the Division of Power System Operations.

By teletypewriter, rainfall reports from U. S. Weather Bureau stations are received from its Knoxville office, and stages on the Ohio River at Paducah and Cairo from the U. S. Weather Bureau at Cairo. During flood periods, observed and predicted stages and discharges for selected stations on the Ohio and Mississippi Rivers are received by teletypewriter from the Corps of Engineers at Cincinnati.

Weather forecasts—Under a cooperative agreement, the U. S. Weather Bureau furnishes TVA with weather forecasts, including quantitative forecasts of precipitation. This service, initiated in 1939, was the first of its kind in this country. The regular forecast, received about 8:00 a.m., gives a specific forecast for the next 36 hours plus an outlook for the succeeding three days. A supplementary forecast is received about 7:00 p.m. during the flood season, giving a specific forecast for the next 36 hours. These forecasts include estimates of the time of beginning and ending of rainfall and quantitative forecasts of precipitation for seven subdivisions of the Valley. Figure 106 is a copy of a typical U. S. Weather Bureau forecast as received by teletype, and figure 107 shows the seven areas for which quantitative forecasts are made.

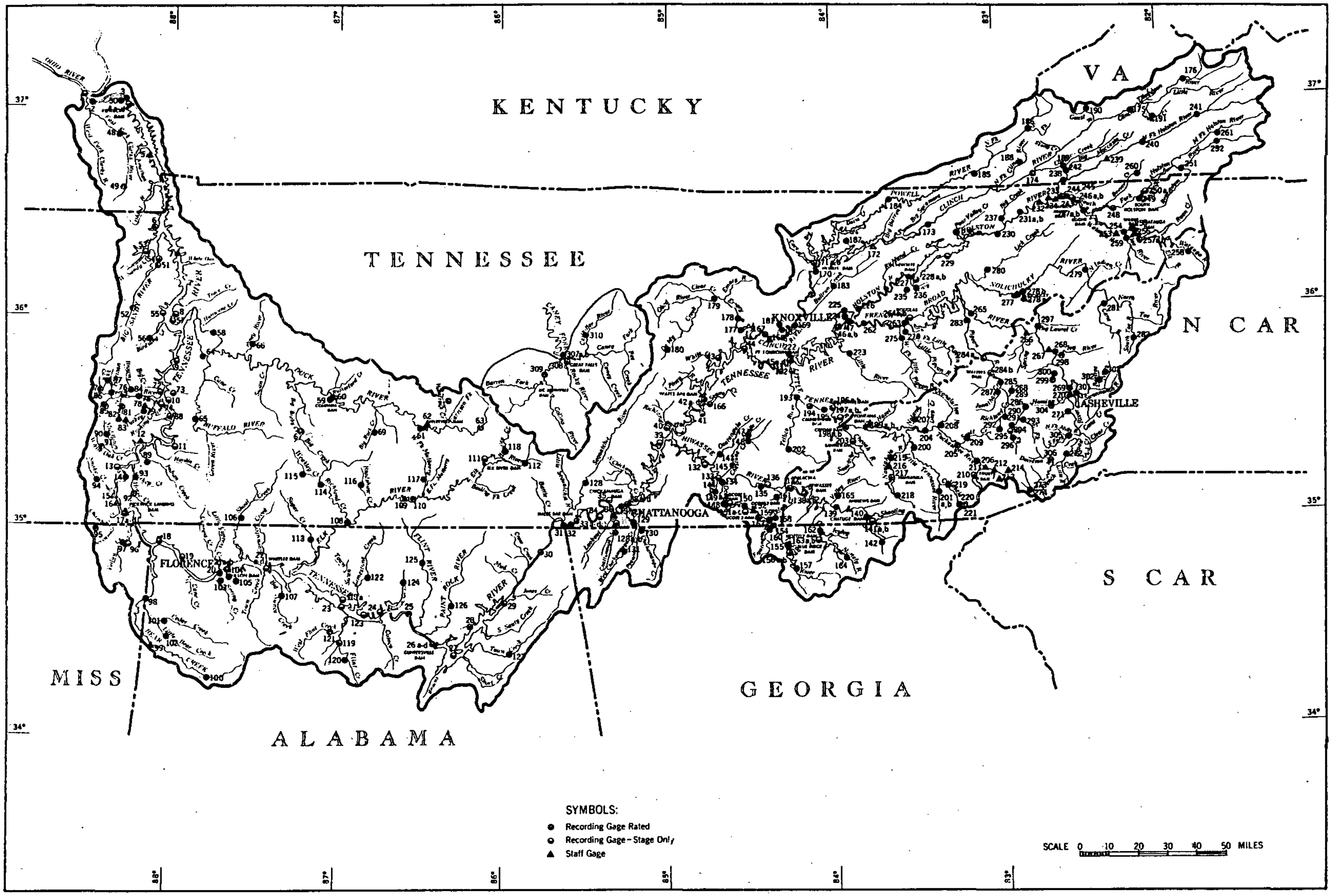


FIGURE 105.—*Stream gages in the Tennessee River Basin—March 1959.*

COPY

REGULAR FORECAST FOR JAN 4 AND 5, 1949

TODAY

OVERCAST CLOUDINESS, INTERMITTENT RAIN AND A FEW SCTD THUNDERSTORMS
COLDER IN THE EXTREME WEST THIS AFTERNOON GENTLE TO FRESH WINDS LIGHT
INTENSITY 4

TONITE AND WEDNESDAY

MOSTLY CLOUDY INTERMITTENT RAIN, EXCEPT NOT MUCH PRECIP ACTIVITY OVER
THE WESTERN THIRD LATE TONITE AND WEDNESDAY GENTLE TO MODERATE WINDS
COLDER OVER THE WESTERN THIRD OF THE AREA TONITE AND OVER THE WESTERN
TWO THIRDS BY WEDNESDAY AFTERNOON LIGHT INTENSITY 4, EXCEPT 3 OVER
THE WESTERN THIRD DURING THE AFTERNOON

PRECIP AMOUNTS DURING THIS FORECAST PERIOD ARE EXPECTED TO AVERAGE AS
FOLLOWS

WESTERN SECTION, .50 OF AN INCH NORTHWEST CORNER UP TO 1.00 TO 1.20 INCHES
SOUTHEAST BORDER
WEST CENTRAL SECTION, 1.00 INCH WEST TO 1.75 INCHES EAST
SOUTHWESTERN SECTION, 1.00 TO 1.20 INCHES WEST UP TO 2.50 INCHES EAST
EAST CENTRAL SECTION, .80 TO 1.00 INCH NORTH AND NORTHEAST BORDER,
RANGING UP TO 2.50 INCHES IN THE CHATTANOOGA, CHARLESTON, AND OCOEE NO 1
AREA
SOUTHEASTERN SECTION, 1.50 TO 2.25 INCHES
EASTERN SECTION, .80 OF AN INCH NORTH TO 1.50 INCHES SOUTH
NORTHEASTERN SECTION, .60 TO 1.00 INCH

THURSDAY, FRIDAY, AND SATURDAY

PRECIP AVERAGING .80 TO 1.20 INCHES WITH HEAVIEST AMOUNTS THRU THE CENTRAL
PORTION OF THE AREA THE MAJOR PORTION OF THE PRECIP IS EXPECTED IN
CONNECTION WITH A CONTINUATION OF THE PRESENT STORM INTO THURSDAY NO
SIGNIFICANT PRECIP IS INDICATED FOR FRIDAY AND SATURDAY

TEMPS DURING THE PERIOD WILL AVERAGE ABOVE NORMAL

SERVICE AREA FORECAST FOR JAN 4 AND 5, 1948

WESTERN TENN AND MISSISSIPPI

CLOUDY WITH INTERMITTENT RAIN TODAY CLOUDY TO PARTLY CLOUDY TONITE AND
WEDNESDAY COLDER LATE THIS AFTERNOON, TONITE, AND WEDNESDAY GENTLE TO
MODERATE WINDS

NORTHERN TENN AND KENTUCKY

CLOUDY WITH INTERMITTENT RAIN TODAY, TONITE, AND WEDNESDAY MILD TODAY,
BECOMING COLDER LATE TONITE AND WEDNESDAY GENTLE TO OCNLY FRESH WINDS

GEORGIA AND ALABAMA

CLOUDY AND MILD WITH INTERMITTENT RAIN TODAY, TONITE, AND WEDNESDAY
GENTLE TO MODERATE WINDS

R J YOUNKIN
1110
EHT
END

FN

FIGURE 106.—*Copy of typical U. S. Weather Bureau forecast as received by teletype.*

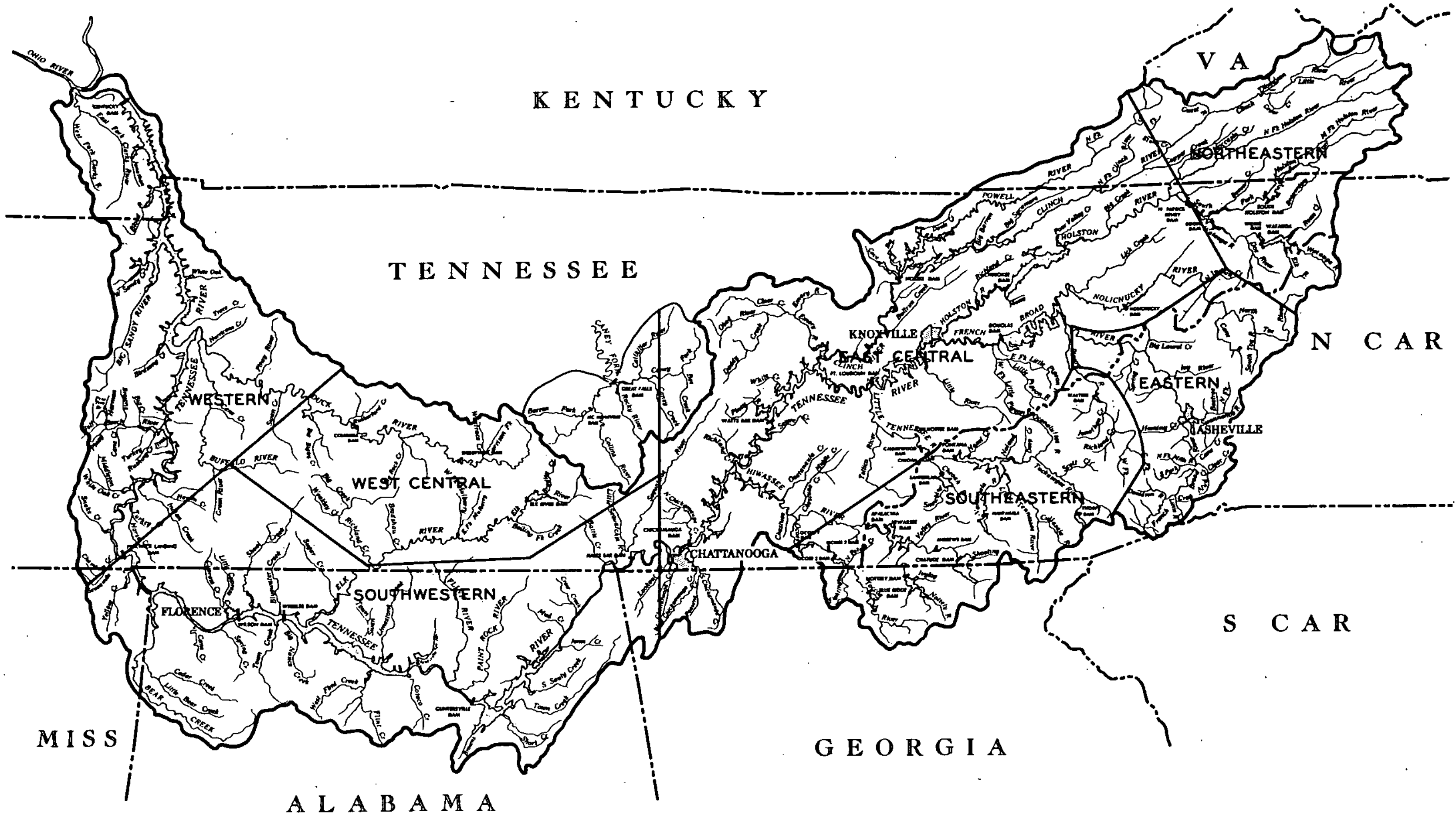


FIGURE 107.—*Tennessee Valley weather forecasting subdivisions.*

During critical periods, Weather Bureau personnel review current reports in the afternoon and evening and determine whether a change in forecast is needed. When floods prevail or when heavy rainfall or other important changes in weather conditions are expected, the local Weather Bureau meteorologist brings the latest weather maps to the TVA office daily and advises the water control engineers of the anticipated weather.

These weather forecasts and consultations have been of great value in operation of the reservoir system. Predictions of the time of beginning of precipitation are helpful in preparing the system for a flood. Predictions of amounts of expected rainfall influence current operations during a flood. Advance knowledge of the time of ending of a storm is very important in determining safe amounts of storage space to be filled in regulation. As the predictions improve in accuracy and reliability, they will become of increasing value in operations.

Reservoir inflow forecasts—Forecasts of reservoir inflows are necessary for planning the operation of the system. To prepare these forecasts requires prompt analysis of data. Incoming data are tabulated on forms. River stages are converted to equivalent stream flow on these forms, which are then reproduced and distributed to the forecasting engineers. Rainfall amounts are plotted on a base map of the Tennessee Valley for use by the engineers. Rainfall averages over each subdivision into which the Basin has been divided for predicting inflows are determined rapidly by use of an electronic digital computer. Figure 108 is a copy of the map prepared early on the morning of January 5, 1949, showing 24-hour rainfall to 6:00 a.m. of that date.

Forecasts of reservoir inflows for 3 to 5 days in advance are made each morning for 33 reservoirs in the Tennessee Valley system and for the Great Falls Reservoir in the Cumberland Valley. To make these forecasts, to schedule water use at each of these reservoirs, to issue river bulletins and flood warnings, and to perform these duties in a limited time requires a moderate-sized staff and planned scheduling of work.

Reservoirs are grouped by drainage areas and assigned to individual engineers. As an example, part of one engineer's assignment is the Holston River reservoirs. This assignment includes making inflow forecasts for South Holston, Watauga, Boone, Fort Patrick Henry, and Cherokee Reservoirs. In addition, he forecasts the effect of scheduled releases from each reservoir at the next downstream plant.

Inflows are computed as volumes to arrive in the reservoirs by calendar days, since continuous hydrographs of inflow are not usually needed for operating purposes. However, continuous observed hydrographs are plotted for reporting streamflow stations on uncontrolled tributary streams for analysis in determining runoff.

Forecast projections are made for major streams for which continuous stage and flow predictions are desired. Figure 109 shows the observed and predicted hydrographs for the Emory River at Oakdale, Tennessee, during the flood of January 1949. Forecasts are usually based on the latest reports of observed rainfall; however, if substantial additional rainfall is predicted, an additional computation of inflow is made for the predicted rainfall. Depending upon conditions, this additional flow may be added to that from the observed rainfall and used in scheduling the current operation, or it may be used separately to compare its effect with a schedule based on observed rainfall.

Reservoir operation schedules—Reservoir operation schedules, specifying daily and sometimes hourly releases from each reservoir in the system, are prepared about noon each day covering the current day and three to five days in advance, depending upon the time required for the flood crest to pass critical locations. Although preparation of this final schedule is not possible until complete observed and forecast data are available, early preliminary changes in discharges must be made during flood periods, particularly at mainstream dams. This is necessary to prevent premature filling of storage space and to avoid the necessity of larger and more rapid discharge changes later. These changes, based on preliminary data, depend upon the judgment by the river control engineers that such changes will be required and will be in line with the final schedule.

In determining the final schedule, the first step is to set tentative daily releases from the tributary reservoirs. These are based on current elevations, inflows, downstream conditions, and weather outlook. These tributary releases are lagged in time for arrival in the mainstream reservoirs. Trial routings of these tributary arrivals plus the mainstream local inflows are then made through the mainstream reservoir system to determine the most desirable operation for Chattanooga and other downstream critical locations. Decisions are made as to the amount of water to be released and the amount of storage space to be filled in each reservoir. These decisions are based on current data and weather outlook. They are checked, as a guide, against the releases and storages which would be called for under the "Fixed rule operation" described in chapter 7 starting on page 134. The effect of the operation on the Ohio-Mississippi Rivers is determined. Figure 110 shows a computation of the routing of flood flows through the mainstream reservoir system which was made on January 15, 1947. When the final operation is decided upon, instructions on discharge changes are transmitted to the Division of Power System Operations for execution at the dams.

During flood periods, additional rainfall observed at noon and again at 6:00 p.m., or a change in the weather forecast, may make it necessary to revise

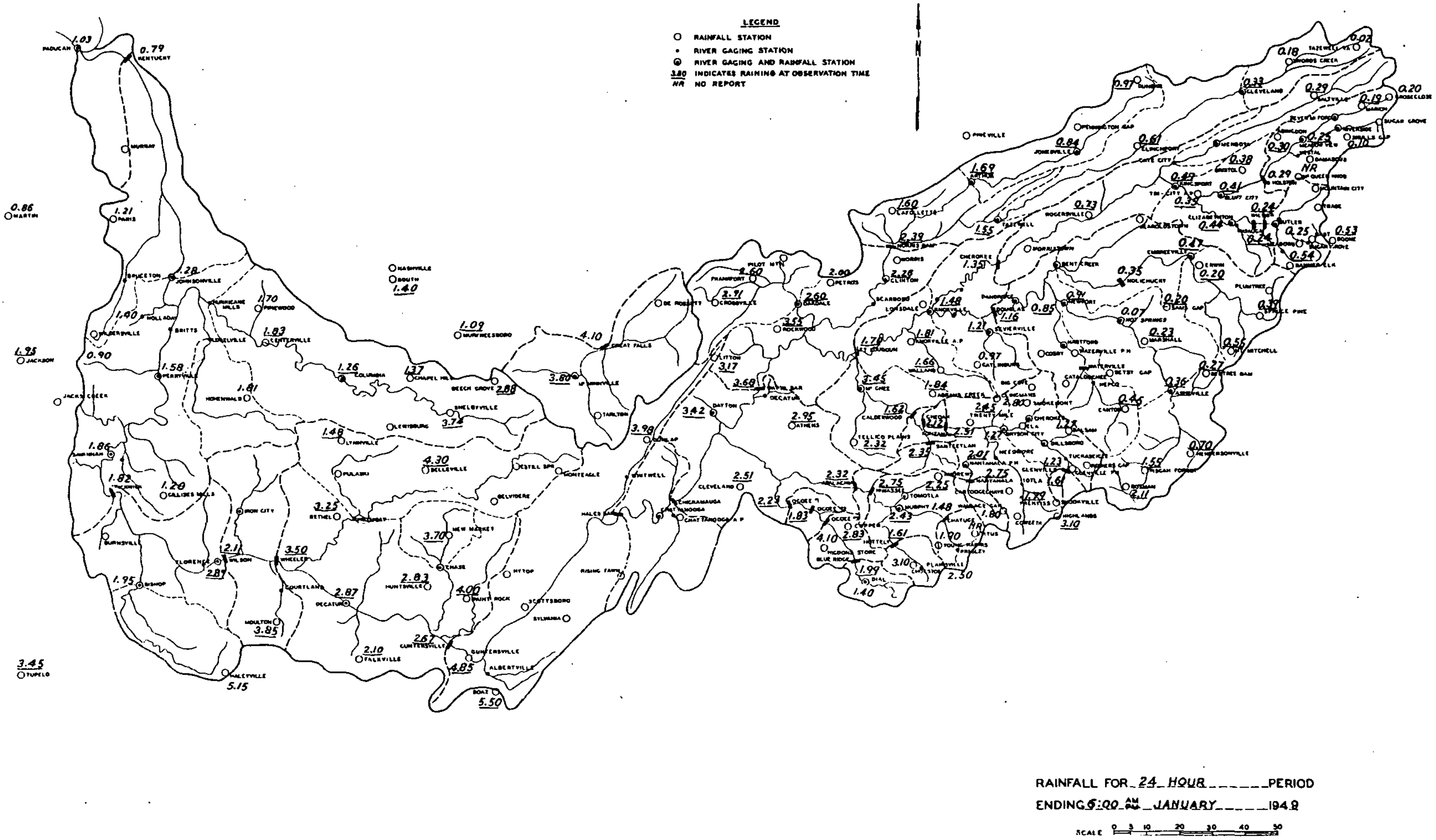


FIGURE 108.—*Map showing daily observed rainfall.*

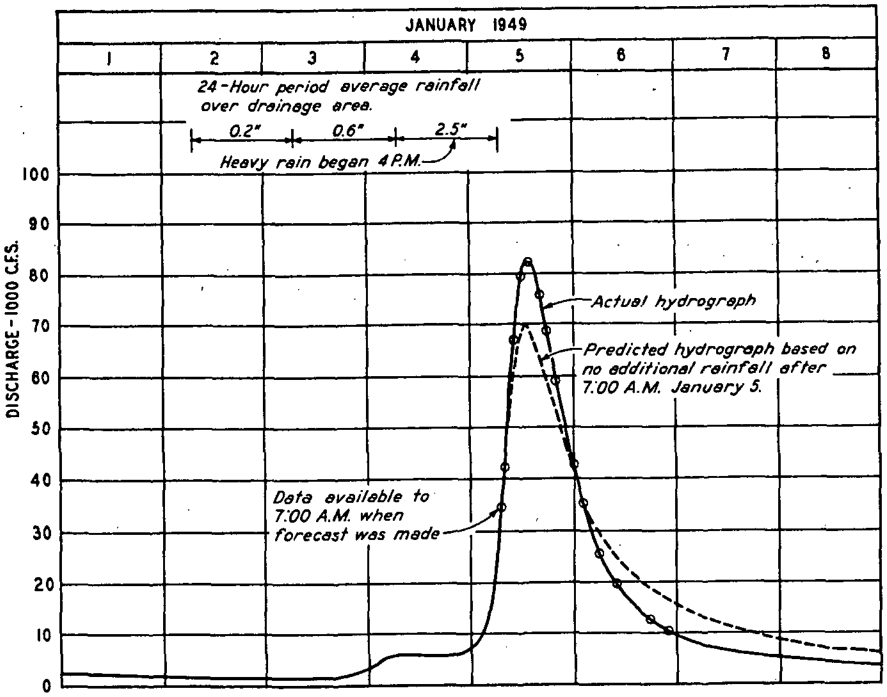


FIGURE 109.—*Hydrographs of Emory River at Oakdale, Tennessee—flood of January 1949.*

the scheduled operation. This may occur several times during the day or night. As the schedule is changed, revised instructions are transmitted on discharge changes at the dams.

Stage forecasts and warnings—Large numbers of persons, both within and outside TVA, are directly concerned with the observed and predicted elevations and flows at TVA reservoirs or at other pertinent locations in the Valley. Timely flood warnings can result in substantial lessening of flood damage. To meet this need, several means of dissemination are utilized. A Daily River Bulletin is issued in the early afternoon in cooperation with the U. S. Weather Bureau giving observed data for the past 24 hours for 48 locations in the Valley and predictions for the following three days. Figures 111 and 112 are copies of the front and back of this bulletin for February 1, 1957. On that date only 47 locations were listed, Chilhowee Dam not having been completed until August 1957.

Observed and predicted stages for about 40 locations are also furnished to the U. S. Weather Bureau for publication by newspapers and for radio broadcast. Special bulletins and warnings are issued for critical locations during flood periods. Information on reservoir operations which will affect the Ohio River is sent by teletype to the U. S. Weather Bureau at Cairo and to the Corps of Engineers at Cincinnati. If the operation schedule is changed during the day or night, new warnings are issued and revised forecasts are furnished to the U. S. Weather Bureau and to the Corps of Engineers.

Maintenance of normal reservoir levels

Under the annual cyclical method used in operating the multiple-purpose reservoirs of the TVA, the so-called normal reservoir level which is maintained in different seasons of the year varies considerably in some cases, as shown by the operating guides, figures 96 and 97, pages 160 and 161. During the flood season, the operating level in tributary reservoirs has a gradually rising trend, which in flood control operations may be, and often is, exceeded temporarily. When this happens, the necessary drawdown to prepare for another flood is accomplished as soon and as rapidly as the surplus water can be discharged downstream with safety.

In the mainstream reservoirs, the operating level is maintained within the "usual winter fluctuation" zone during the flood season, except during flood control operations, when temporary storing of floodwater causes the level to rise. However, the subsequent withdrawal of water restores the water surface to its original elevation as soon as practicable so that all flood storage space may be available in case of another flood.

Figures 113 and 114 show the filling effect of flood storage operations in Douglas tributary reservoir and Chickamauga mainstream reservoir, respectively, during floods in January and February 1946

RESERVOIR ROUTING COMPUTATIONS—VOLUMES IN 1000 DAY-SECOND-FEET

Reservoir	Day	Obs. Data T	Prediction based on observed rainfall to 6:00 am Jan. 15 W	T	F	S	S		3:00 pm Revision — Computation based on predicted rain developing and maintaining discharges set at noon W	T	F	S	4:45 pm Revision increasing releases to lower headwaters W	T
	Date Jan. 1947	14	15	16	17	18	19		15	16	17	18	15	16
	Cherokee	4.4	4.2	4.1	4.1	4.1	4.1		4	4	4	4	4	4
	Douglas	6.1	6.1	6.1	6.1	6.1	6.1		6	6	6	6	6	6
FORT LOUDOUN	Local Inflow	3.1	11.5	10.0	5.5	4.0	3.0	observed rain plus 1.00" additional	25	21	10	6	25	21
	Total Inflow	13.9	21.8	20.2	15.7	14.2	13.2		35	31	20	16	35	31
	ΔStorage	-1.0	-0.2	-5.8	-0.3	0	0		+13	+1	-10	-10	+11	-4
	Storage (Level)	144.5	144.3	138.5	138.2	138.2	138.2		157	158	148	138	155	151
	Elevation 12 pm	807.97	807.9	807.0	807.0	807.0	807.0		809.9	810.1	808.5	807.0	809.6	809.1
	Outflow	14.9	22.0	26.0	16.0	14.2	13.2		22	30	30	26	24	35
WATTS BAR	Local Inflow	11.3	15.7	16.4	10.6	7.4	6.0	observed rain plus 1.00" additional	35	38	20	11	35	38
	Calderwood -1 day	3.6	3.6	4.0	3.8	3.8	3.0		4	4	4	4	4	4
	Norris -1 day	4.4	3.8	4.2	4.2	4.2	4.2		4	4	4	4	4	4
	Total Inflow	34.2	45.1	50.6	34.6	29.6	26.4		65	76	58	45	67	81
	ΔStorage	+4.7	-2.9	-11.4	-0.4	-0.4	-3.6		+17	+4	-14	-24	+13	-9
	Storage (Level)	70.8	67.9	56.5	56.1	55.7	52.1		88	92	78	54	84	75
	Elevation 12 pm	736.42	736.2	735.5	735.5	735.5	735.4		737.4	737.6	736.9	735.4	737.2	736.5
	Outflow	29.5	48.0	62.0	35.0	30.0	30.0		48	72	72	67	54	90
CHICKAMAUGA	Local Inflow	5.1	16.5	13.5	9.0	6.5	~~5.5~~	Observed rain plus 1.00" additional	33	24	14	9	33	24
	Apalachia -1 day	2.0	1.3	1.8	1.8	1.8	1.8		1	2	2	2	1	2
	Ocoee -1 day	1.1	1.1	3.3	6.8	3.4	2.0		1	3	7	3	1	3
	Total Inflow	37.7	66.9	80.6	52.6	41.7	39.3		83	101	95	86	89	119
	ΔStorage	-1.6	+6.9	-9.4	-10.4	-5.3	+0.3		+23	+1	-5	-14	+26	-6
	Storage (Level)	205.4	212.3	202.9	192.5	187.2	187.5		228	229	224	210	231	224
	Elevation 12 p.m	676.36	676.9	676.2	675.8	675.5	675.5		678.0	678.1	677.7	676.7		
	Outflow	39.3	60.0	90.0	63.0	47.0	39.0		60	100	100	100	63	125
HALES BAR	Local Inflow	2.6	8.7	9.6	6.9	5.0	4.4	Observed rain plus 1.00" Additional	14	16	10	6	14	16
	Total Inflow	41.9	68.7	99.6	69.9	52.0	43.4		74	116	110	106	77	141
	ΔStorage	+0.6	+6.7	+2.6	-6.1	-5.0	-1.6		+7	+7	0	-2	+7	+10
	Storage (level)	66.6	73.3	75.9	69.8	64.8	63.2		74	81	81	79	74	84
	Elevation 12 pm	630.7	633.0	634.0	632.0	630.5	629.8		633.0	635.0	635.0	634.6	633.0	636.0
	Outflow	41.3	62.0	97.0	76.0	57.0	45.0		67	109	110	108	70	131
GUNTERSVILLE	Local Inflow	12.2	28	16	11	8	7	Observed rain plus ½" additional	38	22	13	9	38	22
	Total Inflow	53.5	90	113	87	65	[illegible]		105	131	123	117	108	153
	ΔStorage	0	+10	+3	-8	-5	-2		+25	+21	+13	+7	+23	+28
	Storage (Profile)	103	113	116	108	103	[illegible]		128	149	162	169	126	154
	Elevation 12 pm	593.27	593.0	592.7	593.0	593.0	593.0		593.5	593.6	594.0	594.3	593.4	593.0
	Outflow	53.5	80	110	95	70	55		80	110	110	110	85	125
WHEELER	Local Inflow	10.9	10	8	6	5	5	Observed rain and no Additional	10	8	6	5	10	8
	Compensation Elk River	3.5	-5 4	6	+3 5	5	4		-5 4	8	8	5	-5 4	8
	Total Inflow	67.9	89	124	109	80	64		89	126	124	120	94	141
	ΔStorage	+12	+4	+14	-1	-15	-7		+4	+16	+14	+10	+4	+16
	Storage (Profile)	427	431	445	444	429	422		431	447	461	471	431	447
	Elevation 12 pm	550.73	550.1	550.2	550.6	550.6	550.5		550.1	550.3	551.0	551.5	550.1	549.7
	Outflow	55.9	85	110	110	95	71		85	110	110	110	90	125
WILSON	Local Inflow	-0.8	-2	-2	-2	-2	-2	observed rain and no additional	-2	-2	-2	-2	-2	-2
	Total Inflow	55.1	83	108	108	93	69		83	108	108	108	88	123
	ΔStorage	-1.3	0	0	0	0	0		0	0	0	0	0	0
	Storage (Level)	34.7	35	35	35	35	35		35	35	35	35	35	35
	Elevation 12 pm	507.51	507.5	507.5	507.5	507.5	507.5		507.5	507.5	507.5	507.5	507.5	507.5
	Outflow	56.4	83	108	108	93	69		83	108	108	108	88	123
PICKWICK	Local Inflow	11.3	9	8	8	7	7	observed rain and no additional	9	8	8	7	9	8
	Total Inflow	67.7	92	116	116	100	76		92	116	116	115	97	131
	ΔStorage	-1.5	-10	-4	-4	[illegible]	+6		-10	-4	-4	-5	-10	-4
	Storage (Profile)	376	366	362	[illegible]	358	364		366	362	358	353	366	362
	Elevation 12 pm	410.05	409.3	409.0	408.5	408.9	409.0		409.3	409.0	408.8	408.6	409.2	408.9
	Outflow	82.7	102	120	120	100	70		102	120	120	120	107	135
KENTUCKY	Local		32	14	8	6	5	observed rain and no additional	32	14	8	6	32	14
	Duck													
	Total Inflow		134	134	128	106	75		134	134	128	126	139	149
	ΔStorage		+46	+19	+8	-14	-35		+46	+19	+8	+6	+51	+34
	Storage	1037	By separate routing computation						By separate routing computation					
	Elevation 12 pm	353.91	353.8	354.0	354.1	354.2	[illegible]		353.8	354.0	354.1	354.2	353.9	354.1
	Outflow	745	88	115	120	120	110		88	115	120	120	88	115
	Tailwater	310.77	311.4	311.0	310.4									
	Note:		Below Hales Bar all volumes are shown to nearest 1000 day-second-feet						All volumes are shown to nearest 1000 day-second-feet					
	~~System Inflow~~													

FIGURE 110.—*Main stream reservoir routing computations—January 15, 1947.*

TENNESSEE VALLEY AUTHORITY

COOPERATIVE FORECAST SERVICE

DAILY RIVER BULLETIN

KNOXVILLE, TENNESSEE, FRIDAY, FEBRUARY 1, 1957

REPORTED RIVER ELEVATIONS AND RAINFALL—TENNESSEE RIVER BASIN

RESERVOIR OR STATION	Miles Above Mouth	Top of Gates or *Flood Stage*	January 31, 1957 ☒ Inflow	Discharge	Elevation	6 A. M. Feb. 1 RAINFALL 24 hrs.	Month to date
SOUTH HOLSTON	50	1742	12,800	1,089	1703.62	1.44	1.44
WATAUGA	37	1975	7,690	15	1922.24	1.55	1.55
BOONE	19	1385	11,300	9,298	1368.08	1.48	1.48
FORT HENRY	8	1263	10,000	9,839	1260.75		
CHEROKEE	52	1075	48,800	6,534	1028.55	1.74	1.74
NOLICHUCKY	46	72	14,300	12,603	76.5	2.22	2.22
DOUGLAS	32	1002	58,000	14,743	955.09	1.90	1.90
THORPE *	10	3100	830	104	3068.71	2.56	2.56
NANTAHALA *	23	2890	4,150	276	2854.96	4.50	4.50
FONTANA	61	1710	41,500	967	1607.68	3.52	3.52
SANTEETLAH *	9	1817	10,200	34	1790.55	2.89	2.89
CHEOAH *	51	1154	2,850	2,689	1153.44	3.30	3.30
CALDERWOOD *	44	965	7,500	7,723	962.92	3.08	3.08
NORRIS	80	1020	82,200	6,780	991.45	1.57	1.57
Oakdale (Emory R.)	18	*788*		19,000	780.8		
CHATUGE	121	1928	3,730	78	1903.90	3.63	3.63
NOTTELY	21	1780	3,610	101	1735.12	3.20	3.20
HIWASSEE	76	1526	17,800	943	1463.47	3.68	3.68
APALACHIA	66	1280	2,200	1,367	1277.88	3.29	3.29
BLUE RIDGE	53	1691	3,470	101	1638.80	3.87	3.87
OCOEE NO. 3	29	1435	4,580	4,221	1434.92	3.49	3.49
OCOEE NO. 1	12	831	9,360	3,338	831.2	3.05	3.05
GREAT FALLS (Cumberland Valley)	91	805	44,100	43,996	805.21	1.80	1.80
Knoxville	645	*817*			811.83	1.96	1.96
FORT LOUDOUN	602	815	48,400	33,700	809.57	2.06	2.06
Fort Loudoun Lower Lock					752.2		
WATTS BAR	530	745	108,000	96,100	736.82	2.39	2.39
Watts Bar Lower Lock					689.3		
CHICKAMAUGA	471	685	134,000	136,500	676.42	2.23	2.23
Chattanooga	464	*651*			649.31	2.32	2.32
HALES BAR	431	635	160,000	147,300	632.55	2.70	2.70
Hales Bar Lower Lock					614.4		
GUNTERSVILLE	349	595	229,000	172,900	593.71	3.00	3.00
Guntersville Lower Lock					572.4		
Decatur †	305	*559*			554.78	2.56	2.56
WHEELER	275	556	278,000	211,200	551.32	2.88	2.88
WILSON	259	508	242,000	235,300	505.89	2.73	2.73
Florence	257	*419*			423.88	2.80	2.80
PICKWICK	207	418	287,000	241,600	410.56	2.56	2.56
Pickwick Lower Lock					388.3		
Savannah	190	*380*			380.40	1.87	1.87
Perryville	135	*378*			365.14	.90	.90
Hurricane Mills (Duck R.) †	26	*386*		41,500	392.4		
Johnsonville	100	*377*			358.45	.75	.75
KENTUCKY	22	375	360,000	281,500	353.53	.24	.24
Kentucky T W		*320*			328.98		
Paducah (USGS Ohio R.) †	0	*325*			322.9		
AVERAGE RAINFALL ABOVE CHATTANOOGA (Approximate)(Mean Feb. rainfall 4.56 in.)						2.15	2.15
AVERAGE RAINFALL BELOW CHATTANOOGA (Approximate)(Mean Feb. rainfall 4.88 in.)						2.01	2.01

☒ Inflow and discharge are average midnight to midnight in cubic feet per second and elevation is at the end of the day except as noted. Inflow is the reported discharge corrected for change in storage. Apparent upstream slope between any two stations may be due to surges or datum differences within a reservoir.

† Elevation and discharge are at about 6:00 a.m. C.S.T. today. (e) Estimated. * Elevations are Alcoa Datum.

WEATHER FORECAST FOR TENNESSEE VALLEY: Rain with not much change in temperature tonight and Saturday. Low tonight 42 to 47 western portion and 47 to 56 eastern portion. High Saturday 45 to 55 western portion and 56 to 65 eastern portion.

TEMPERATURES	Max. Yesterday	Min. Last Night	TEMPERATURES	Max. Yesterday	Min. Last Night
Asheville	59	51	Fontana	55	51
Birmingham	75	59			
Bowling Green	44	39	Knoxville	56	53
Bristol	52	50	Memphis	41	40
Chattanooga	62	55	Nashville	49	45
Florence	54	50	Shawnee S. P.	36	30

TVA 1333 (WCP-10-53) U. S. WEATHER BUREAU

FIGURE 111.—*Front of Daily River Bulletin.*

PREDICTED RIVER FLOWS AND ELEVATIONS

Predicted elevations and discharges are based on stream flows and rainfall reported up to 6:00 a.m. of the date of issue and on scheduled releases from TVA and Alcoa Dams. Releases are subject to change without notice in case unpredictable conditions so require. Elevation forecasts are for midnight C.S.T. at the end of the day and inflows and discharges are average midnight to midnight in thousand cubic feet per second. Predicted River Flows and Elevations given in this bulletin must not be used, in whole or in part, for broadcasting or publication after 8 a. m. on the day following the date of publication.

RESERVOIR OR STATION	February 1			February 2			February 3		
	Inflow	Discharge	Elevation	Inflow	Discharge	Elevation	Inflow	Discharge	Elevation
SOUTH HOLSTON	16.00	2.60	1708.0	8.60	2.60	1709.9	4.60	2.60	1710.5
WATAUGA	8.80	0	1925.7	4.50	0	1927.5	2.70	0	1928.5
BOONE	16.90	11.00	1371.6	8.90	11.00	1370.3	6.30	11.00	1367.4
FORT HENRY	11.90	11.90	1260.8	11.40	11.40	1260.8	11.20	11.20	1260.8
CHEROKEE	47.20	16.20	1032.6	34.90	16.20	1034.9	23.40	16.20	1035.7
NOLICHUCKY	36.60	37.10	75.6	16.90	18.00	73.1	6.40	6.80	72.1
DOUGLAS	118.00	14.50	967.1	83.00	14.50	974.0	39.00	14.50	976.2
THORPE *	0.70	0	3070.1	0.35	0	3070.8	0.25	0	3071.2
NANTAHALA*	2.60	0	2859.1	2.00	0	2862.3	1.70	0	2864.9
FONTANA	61.22	0	1627.9	22.90	1.00	1634.4	10.50	1.00	1637.2
SANTEETLAH *	6.00	0	1796.2	4.00	0	1799.8	3.00	0	1802.3
CHEOAH *	1.20	1.34	1153.0	1.50	1.50	1153.0	1.35	1.35	1153.0
CALDERWOOD *	4.44	4.94	961.0	2.50	2.50	961.0	1.95	1.95	961.0
NORRIS	68.00	8.50	996.2	59.00	8.50	1000.1	41.00	8.50	1002.4
CHATUGE	3.67	0	1905.7	1.76	0	1906.5	1.20	0	1907.1
NOTTELY	3.02	0	1738.6	1.39	0	1740.2	1.04	0	1741.3
HIWASSEE	19.52	0.69	1476.1	8.06	1.34	1480.2	4.71	2.75	1481.3
APALACHIA	2.33	2.80	1277.0	1.80	3.00	1275.0	3.04	3.00	1275.1
BLUE RIDGE	5.00	0	1644.3	2.70	0	1647.1	1.30	0	1648.4
OCOEE NO. 3	6.20	6.20	1435.0	3.00	3.00	1435.0	1.50	1.50	1435.0
OCOEE NO. 1	14.80	14.00	832.0	5.90	9.90	830.0	2.50	3.30	829.2
GREAT FALLS	65.00	65.00	805.2	31.00	31.00	805.2	17.00	17.00	805.2
Knoxville			813.0			811.5			810.0
FORT LOUDOUN	55.0	73.0	810.6	22.0	55.0	810.2	10.0	50.0	808.6
Fort Loudoun L.L.			748.5			747.0			746.0
WATTS BAR	198.0	150.0	739.5	118.0	118.0	739.5	83.0	100.0	738.5
Watts Bar L.L.			692.5			687.5			687.0
CHICKAMAUGA	210.0	180.0	678.6	182.0	180.0	678.7	131.0	170.0	677.0
Chattanooga			652.6			651.6			649.0
HALES BAR	214.0	200.0	632.5	202.0	202.0	632.5	182.0	193.0	632.5
Hales Bar L.L.			616.5			616.5			614.0
GUNTERSVILLE	297.0	240.0	594.5	241.0	260.0	593.8	213.0	230.0	593.5
Guntersville L.L.			577.5			578.5			576.5
Decatur			557.2			558.2			557.3
WHEELER	378.0	278.0	552.7	392.0	310.0	554.8	319.0	330.0	555.0
WILSON	321.0	328.0	505.0	340.0	340.0	505.0	340.0	340.0	505.0
Florence			426.0			426.2			426.4
PICKWICK	403.0	340.0	413.6	388.0	360.0	415.1	371.0	360.0	415.7
Pickwick L.L.			392.7			393.8			394.8
Savannah			384.3			387.5			389.4
Perryville			367.3			369.5			370.8
Johnsonville			359.1			359.6			359.9
KENTUCKY	469.0	310.0	353.0	451.0	325.0	353.0	433.0	325.0	353.0
Kentucky T W			332.1			333.1			333.6
Paducah (USGS Ohio R.) ††			325.1			326.3			327.1

RIVER NOTICE

L. R. ENGSTROM
River Forecaster

†† Stage prediction for 7 a.m. on following day

IMMEDIATE

TVA 1335A (WCP 11-56)

United States Weather Report

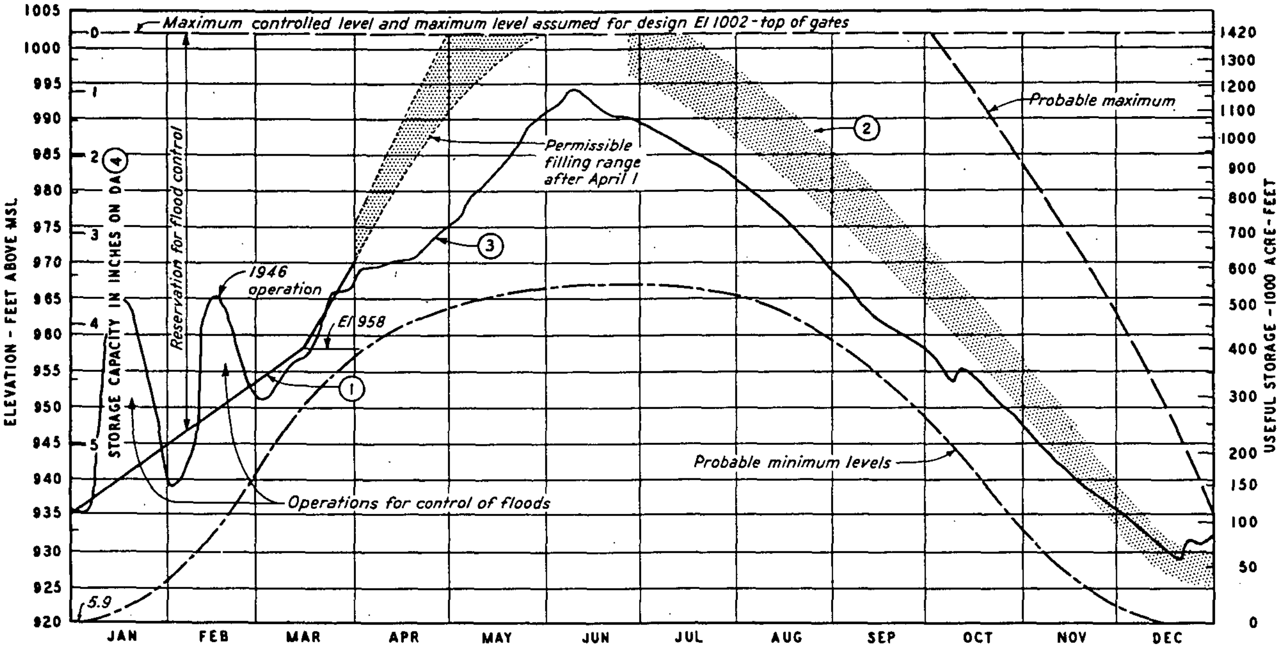
U. S. Department of Commerce
WEATHER BUREAU
Official Business

FIGURE 112.—*Back of Daily River Bulletin.*

and the subsequent restoration of normal flood-season levels. The normal flood-season level in both tributary and mainstream reservoirs is maintained by making use of as much of the water as possible through the turbines, but much of it may have to be wasted, particularly on the mainstream during and after the passage of a flood.

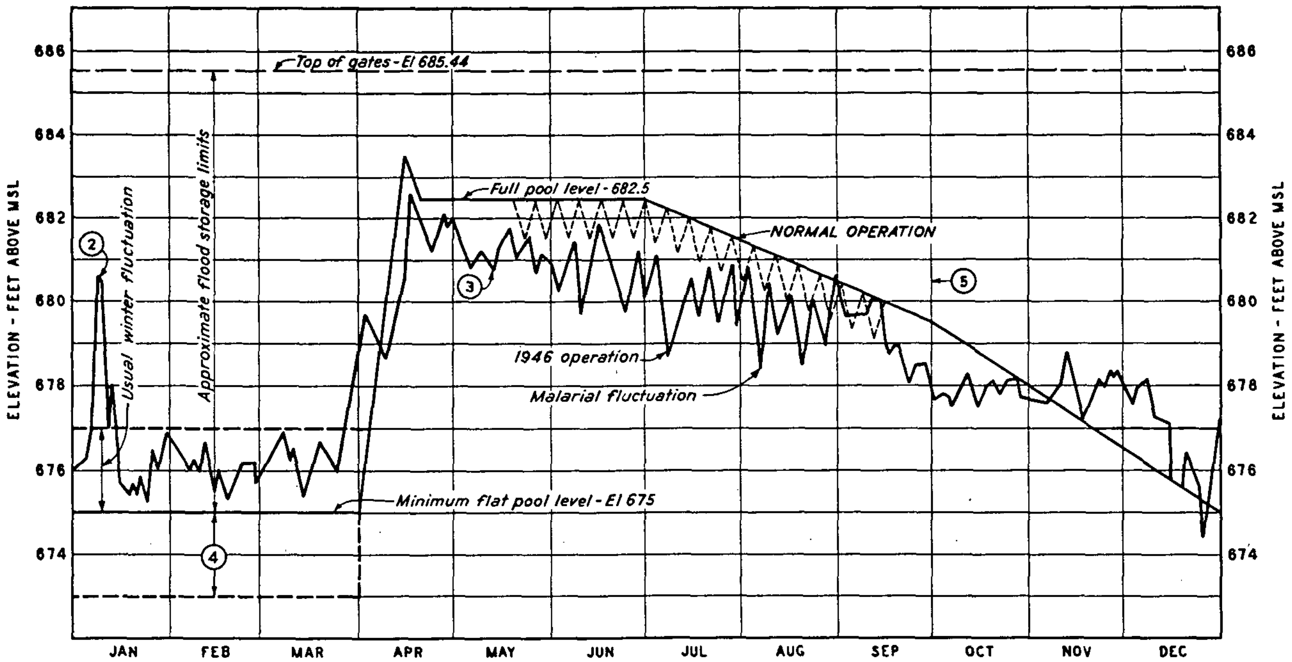
Reservoir routing

To determine the elevations in the reservoirs resulting from a given discharge or the discharge necessary to obtain a given elevation, it is necessary to route inflows through the reservoirs. The TVA tributary reservoirs have large volumes of storage



① Maximum multipurpose levels during flood period to be exceeded only during flood control operations.
② Normal type operation during drawdown period.
③ Failure to complete filling in 1946 due to spillage in January and February for flood control operations and subsequent dry spring.
④ Based upon drainage area, 4,541 square miles.

FIGURE 113.—*Actual 1946 operation—Douglas Reservoir.*



① Elevations apply only at dam.
② Filling and subsequent restoration of normal flood season level during flood control operation.
③ Pool was held about 1½ feet below summer normal to aid dredging.
④ Drawdown zone for maintaining flat pool volume prior to flood crest may extend to El 673 at dam.
⑤ The pool may be raised as high as El 682.5 after October 1 for power storage, but after December 1 it must be kept within the winter fluctuating range.

FIGURE 114.—*Actual 1946 operation—Chickamauga Reservoir.*

compared to rate of flow so that for all practical purposes they can be considered level pools, and routing is accomplished by using level storage tables. In the mainstream reservoirs, however, flood flows are large in relation to the limited channel capacities in the upper reaches of the reservoirs. The volume of water held temporarily in storage between level pool and the backwater profiles becomes appreciable and must be accounted for as it goes into storage on rising flows and out of storage on falling flows. This volume is a function of the flow throughout the reservoir and the headwater elevation at the dam. The method used in routing is dictated in part by the necessity for speed and requires a simple procedure.

Studies of profiles and resulting storages in past floods indicate that the elevation of a point in the upstream reaches of a reservoir in combination with the simultaneous headwater elevation of the reservoir at the dam can be used to define approximately the total volume of storage in the reservoir (fig. 115).

Current natural flow forecasts

Evaluation of the effect of a TVA flood control operation requires a knowledge of the flows and stages which would have occurred without the existence of the system of multiple-purpose reservoirs. Current knowledge of this hypothetical flood is helpful in the actual flood control operation in maintaining awareness of the magnitude of the flood. It is used at times as a guide in limiting releases to avoid exceeding stages which would have occurred under pre-reservoir conditions. Current forecasts of natural flows and stages are made during flood periods.

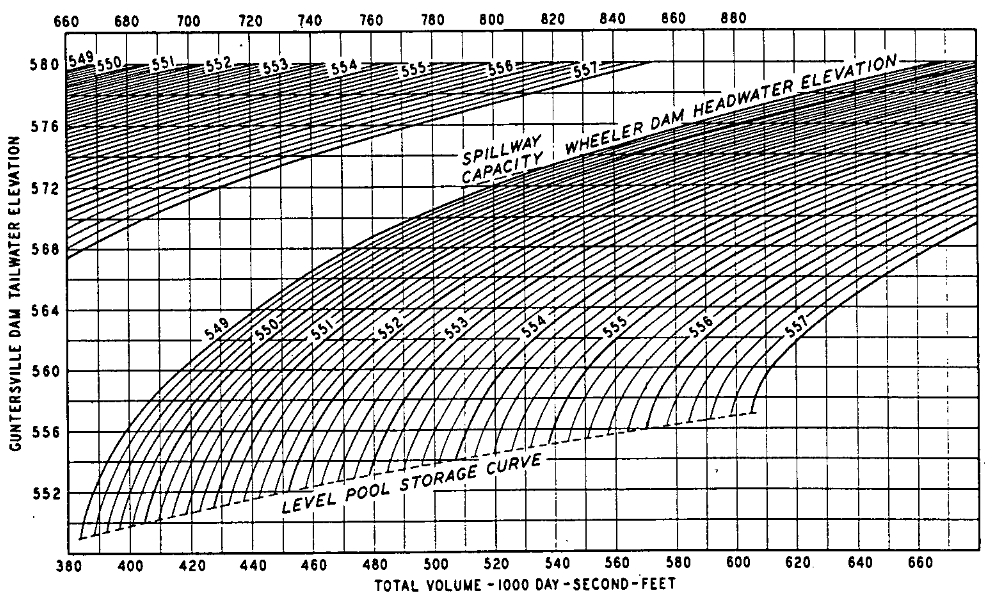
Flood control operation—example

Although the basic concepts of flood control operation can be established by recognition of objectives and study of past experience, the operation of a large, multiple-purpose reservoir system cannot be reduced to fixed routine procedures to be followed for all floods. No two floods are exactly alike. Distribution, duration, and intensity of storms vary; available space is never identically distributed; and downstream requirements change. It is necessary to deviate in the day-to-day pattern of control followed from one flood to another, while still adhering to the same basic principles.

Appendix A gives a day-to-day account of flood control operations during the flood of January 1947. Although the flood of January-February 1957 had greater volume and the peaks were higher than in 1947, the distribution of the 1957 runoff was, in general, much more uniform, and therefore 1947 presented more problems. Hence, the latter is used to illustrate the details of an important operation.

PROBLEMS AFFECTING OPERATION FOR FLOOD CONTROL

Changing the flow of a stream—either increasing low flows or reducing flood flows by means of reservoirs—presents a wide variety of problems, most of which are due to public reaction to any change of whatever nature, even though it may be beneficial.



These curves were developed from observed reservoir water surface profiles through January 1947. Total volumes were determined from five-mile reach volume curves developed by the River Control Branch and dated August 11, 1939, adjusted to published level volumes.

FIGURE 115.—*Volumes under backwater profiles—Wheeler Reservoir.*

These problems, which are of both temporary and continuing nature, are discussed in the following paragraphs.

Temporary problems

During many flood control operations of the system, it has become desirable to protect and facilitate construction work and other miscellaneous activities along and in the river by limiting the discharge temporarily; to minimize flooding of crops, property, or lands on which livestock are pastured, such as islands below dams; and to assist in numerous other emergencies. Other problems which may indirectly affect flood control operations involve making increases or decreases in discharges or fluctuations in pools to aid in refloating grounded or sunken river craft, or maintenance work on power generating units; rearranging gate spillage to aid navigation in entering locks; and raising reservoir elevations to aid in launching boats. However, all such special operations have been subordinated to the over-all plan of operation for maximum flood control benefits.

Continuing public problems

One of the most difficult problems constantly facing water control engineers is the misunderstanding of flood control by the public. In spite of newspaper items, talks, legal decisions, magazine articles, and books covering the subject, the average individual cannot understand that he is not being damaged by the operation of the reservoirs when he sees water over his fields and gates still open in the dam upstream. It is economically unsound, of course, and usually physically infeasible to provide reservoirs which can store the entire flood. During a flood, considerable spillage is absolutely necessary from the mainstream reservoirs which, except Kentucky Reservoir, have relatively small flood storage space. However, the maximum regulated flow from each reservoir is usually less than the crest flow which would have prevailed without the existence of the upstream multiple-purpose reservoirs. The resulting regulated crest stage, even with backwater conditions, is correspondingly lower than the natural crest stage would have been (appendix B).

Another major problem is that as the ordinary highwater levels which would have occurred downstream under natural conditions are reduced by reservoir operation for flood control, people push their activities—even building homes—down to the new regulated levels. They refuse to believe that when a flood of major proportions occurs even the regulated flow below the dam will be relatively so high that it will cause serious damage in their new zone of activity.

After Norris was first completed, releases of 25,000 cubic feet per second or more caused no downstream damage. Today, a release of 12,000 cubic feet per second, only a little higher than turbine capacity, produces many damage complaints. In a flood having an inflow of 200,000 cubic feet per second, a release from the reservoir of around 40,000 cubic feet per second will be unavoidable. The results will be damaging, and yet there seems to be no workable solution to this problem. Its counterpart is that after a reservoir has not been completely filled for a few years, the utmost vigilance is required to prevent persons from building at or even within the storage reservation, although they have no such right.

It is obvious from an inspection of a flood hydrograph that if the natural crest flows are reduced by upstream storage, flows at other times will be increased when the stored water is released. Determination of the most opportune time to release the stored water is one of the problems in reservoir operation. The ideal method requires the holding of stored water until natural flows again become low, but such low flows may not occur until six or more months later. Successive large storms may occur within this period and, with storage space insufficient to hold a succeeding flood, the reservoirs might be ineffectual. Thus, stored water must be released soon after the flood, with resulting continued high stages downstream. This is particularly true with the main-river reservoirs because of their limited storage. As a result of these continued high stages, some property owners along the river are inconvenienced, but not nearly as much as they would have been by the unregulated flood crest. The continued high stages also may reduce the power head, resulting in a smaller power output.

The more rapid rise in stages below main-river dams than would have occurred under natural conditions is another cause of many complaints and claims by farmers, barge and dock operators, and industries. It results from holding the headwater at the dams to a constant level, or from lowering it in the early part of a flood, thus preventing as much water from going into storage as would be the case in the natural state. Even a minor rise in headwater may not give a storage increment equal to that for natural conditions. This advance, or acceleration, of the flood is largely unavoidable if the reservoir storage is to be saved for use during the crest period. Although the resulting rapid rise in stage may cause some inconvenience, it is highly beneficial in the regulation of almost all Ohio and Mississippi River floods. It seems to be a question of becoming accustomed to the changed conditions. Warnings of approaching flood stages are broadcast by radio from current rainfall and runoff predictions as long in advance as possible.

Complaints and claims received from occupants of land on which TVA owns flooding easements often result from a lack of knowledge of the easement provisions, both as to limiting elevations and dates. On some land along the Tennessee River, easements were purchased giving the right to flood any time in the year; on other land, 6-month flooding easements from December 1 to June 1 were purchased; and on still other land, the fee was purchased. The original grantors of the flowage easements often have forgotten the deed provisions which were carefully explained to them at the time of TVA purchase, or later buyers of these lands subject to TVA flowage rights often have not read their deeds, and some owners are influenced by false rumors. In the upper reaches of Kentucky and Wheeler Reservoirs some occupants of land on which TVA owns flooding rights believe that there will never again be floods above the easement or fee purchase elevation. This, of course, is not correct, as there will be many floods in the future, as there have been in the past, that will exceed these elevations. The limiting purchase elevations were determined from consideration of both the beneficial effect of upstream regulation and the adverse effect of backwater from the downstream dam. The adopted limiting elevations were those below which flooding, on the average, would be increased and above which it would be decreased. Every feasible means of informing the landowners and tenants of the correct easement and fee purchase elevations and effective flood season dates, where applicable, has been employed, such as conferences with the county farm agents and community meetings.

Some people who own or farm land along the lower Tennessee River in areas that are inundated during flood control operations believe that such operations cause the flooding and consequent damage to their crops, livestock, or other property. This belief may be the result of a misunderstanding of the flooding easements owned by TVA, the exact location of purchase contours on their land, and a lack of knowledge of the amount of flood reduction which can be accomplished on this section of the river by the reservoirs. All claims for alleged damages are investigated carefully to determine the facts about the cause of the flooding.

A problem common to all flood control is the encroachment of new development into areas which are given only partial protection. An example of this encroachment is in Chattanooga, where there is some new construction on land which formerly was flooded frequently. Here, the TVA reservoir system has reduced a number of large floods from 10 to 22 feet. Other floods occurring before completion of the present reservoir system were reduced by lesser amounts. Without the necessary additional protection against the maximum probable flood this land is still subject to flooding, although less frequently than heretofore. Although such new construction shows the high degree of confidence placed in the reservoir control, if continued to still lower areas it would result in a flood hazard to the new development approaching, if not equaling, the hazard to properties at considerably higher elevations prior to the construction of the reservoir system. Until the additional protection is provided, further development to these lower elevations should be discouraged by every possible means.

CHAPTER 10

LOCAL FLOOD PROBLEMS IN THE TENNESSEE RIVER BASIN

Before discussing local flood problems in the Tennessee River Basin, this chapter outlines the principal objectives of flood control and describes several alternative methods of preventing flood damage. It then briefly summarizes TVA's preparation of reports to aid Valley communities solve flood problems not taken care of by the reservoir system, after which the flood problems and possible methods of reducing damage at many Tennessee Basin communities subject to damaging floods are discussed in detail.

PRINCIPAL OBJECTIVES OF FLOOD CONTROL

On thousands of acres formerly subject to flooding under natural conditions, the TVA reservoirs have either completely eliminated damages from floods as great as the maximum known or have appreciably reduced the depth or frequency of flooding. These lands lie principally along the upper reaches of the main-river reservoirs, with appreciable amounts along the lower reaches of the tributary streams downstream from multiple-purpose reservoirs. The operation of the reservoirs during a flood, however, is directed principally at reducing crest stages (1) at Chattanooga, (2) below each of the Tennessee River dams, and (3) on the lower Ohio and Mississippi Rivers. In the case of Watauga and South Holston Reservoirs, the focal points of protection are the cities of Elizabethton and Kingsport.

The degree of protection provided by the reservoirs varies with the distance from the large tributary storage reservoirs, almost complete protection against the maximum known flood being given immediately below each of them. Such a large reduction, however, is not possible below the main-river dams. At Chattanooga a reduction in crest discharge of about one-third may be expected in most damaging floods. The corresponding reduction in crest stage will depend on the size of the flood, but will be about 14 feet in the maximum known flood modified to assume an adverse distribution of the storm rainfall with respect to the uncontrolled area.

Crest stage reductions become smaller as the Ohio River is approached because of (1) the longer distance from the storage reservoirs, (2) the greater capacity of the stream channel, and (3) the small storage capacity of the main-river projects relative to the great increase in the size of their drainage basins. Reductions up to 2 or 3 feet in crest stage may be expected on the lower Ohio and Mississippi Rivers through operation of the TVA reservoir system when the Tennessee River is itself in flood.

Cities in the Tennessee Basin where flood damage has been substantially reduced by the present TVA reservoirs include Chattanooga, Knoxville, Dayton, Loudon, and Lenoir City on the Tennessee River; Clinton on the Clinch River; Elizabethton on the Watauga River; and Kingsport on the South Fork of the Holston River. At some of these towns the flood hazard has been largely eliminated, while at others supplemental protective works will be required before complete protection against the maximum probable flood is attained. The situations at Chattanooga and at Kingsport are examples of cities in this latter class.

Flood hazards or problems from small streams outside the influence of the reservoirs still exist, and they are discussed later in this chapter under "Location of Flood Problems in Tennessee Basin."

ALTERNATIVE METHODS OF PREVENTING FLOOD DAMAGE

Property may be protected from floods by some form of physical construction, such as levees and walls, channel improvements, and reservoirs. With this type of protection, no change is made in the property itself or its location. A second type of flood damage prevention is the protection of property on an individual, separate basis, such as the relocation or floodproofing of structures. Flood-warning systems to give time for removal of damageable property would fall in this category. A third type is to control the use and occupancy of the flood plain. A fourth type is the development and conduct of educational and action programs for securing land cover that is consistent with the objectives of water control, watershed protection, and optimum farm income.

Any one of the methods, or any combination of them, may prove to be the most economical in a given situation. Dividing the needed protection between two or more methods may prove cheaper than protection by a single method. For example,

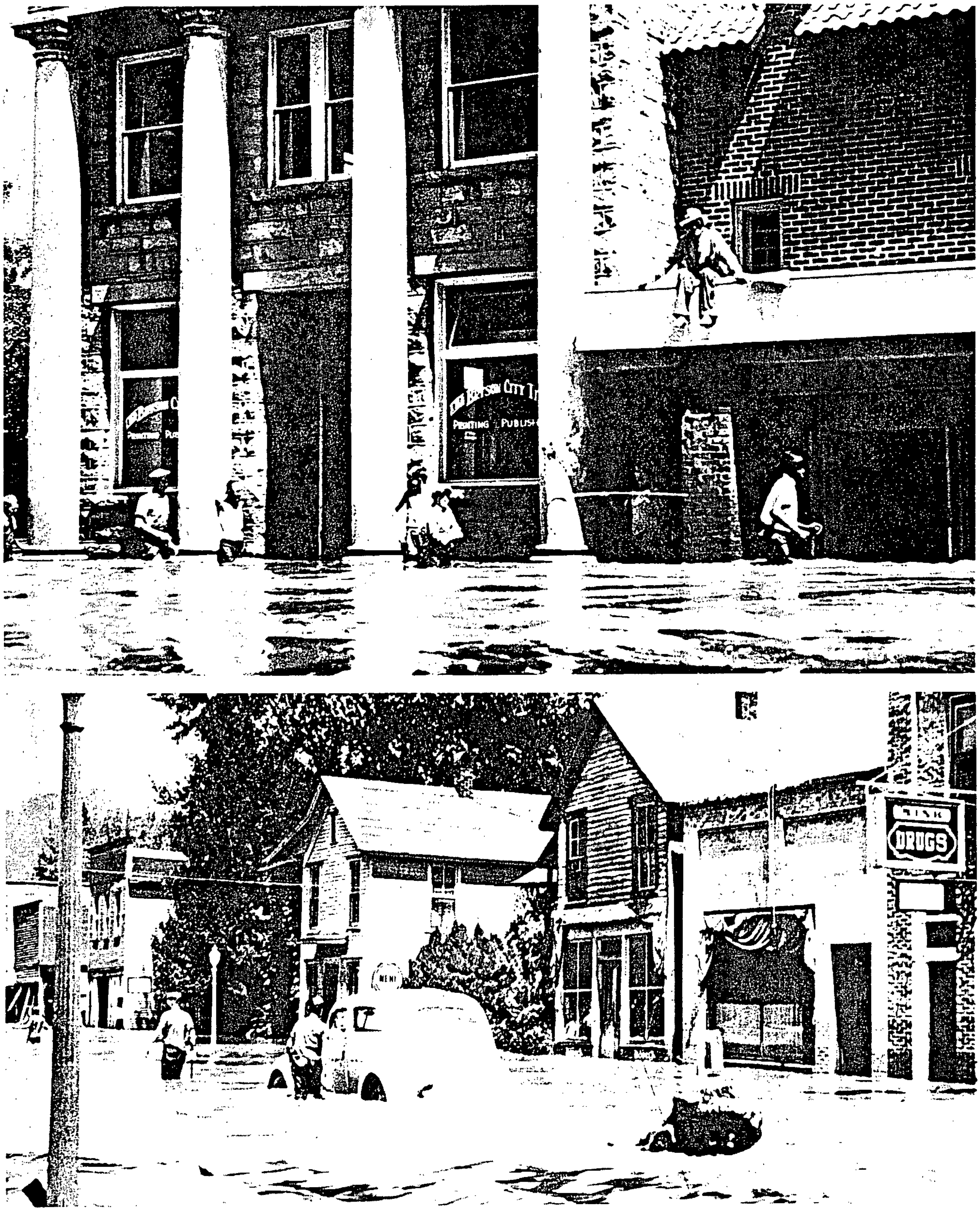


FIGURE 116.—*Street scenes in Bryson City, North Carolina, during flood of 1940. Top: The flood crest was over 2 feet higher than water level in picture. Bottom: At crest stage the water was near the top of the fenders on the automobile.*

in reservoirs both the volume of water to be stored and the cost of its storage space usually increase rapidly for each additional unit of downstream flood-stage reduction. Likewise, the cost of levees and rights-of-way in urban areas also increases rapidly with height, so that it may prove cheapest to provide partial flood reduction by a moderate amount of storage and to confine the flood so reduced between levees of moderate height. Partial reservoir protection may also be combined with zoning or removal below the protected zone.

Following is a brief discussion of the several methods of flood damage prevention mentioned above (except the fourth type which is covered in the next chapter) and of their applicability in the Tennessee Valley.

Reservoirs

The use of reservoirs for storing floodwater to reduce downstream flood elevations has increased greatly in the past 40 years. Their practicability depends largely on the availability of feasible upstream dam and reservor sites of sufficient capacity to control a substantial part of the area. Such reservoirs substitute deep, controlled flooding on land reserved for that purpose for uncontrolled flooding of valuable downstream property (fig. 117). Flood control reservoirs may be classified as to whether the discharge outlets are controlled by gates or have no gate control.

The latter type, called detention basins, discharges at almost a uniform rate during a flood. This rate of discharge is determined in advance and usually is about equal to the channel capacity downstream.

The amount of storage space to be provided in any reservoir depends on several factors, but mostly on the size of the flood against which protection is to be given, the capacity of the channel at the flood-damaging stage, and the relative area controlled above the point of hazard. Generally, a storage equivalent to between 6 and 12 inches of water over the drainage area is sufficient for controlling most floods in the Tennessee River Basin.

The Congressional directive stipulated that TVA construct dams and reservoirs of a type to produce the maximum benefit for navigation and at the same time to contribute to the control of destructive floodwaters in the Tennessee River and Mississippi River. This joint requirement of the Act was fulfilled by using high dams and large reservoirs which would provide large amounts of floodwater storage. Small upstream detention reservoirs would not satisfy the requirement and have not been used in TVA's flood control program.

Levees

Levees or floodwalls are a positive means of holding floodwater off the land and have been used extensively (fig. 118). The confinement of the flow within a levee may raise the height of floods by elimination of valley storage and flow area; therefore, care must be taken to allow for this increased height in the construction of the levee. Levees introduce sanitary and storm drainage problems on the protected area, frequently requiring pumping systems. Roads, drains, and railroads passing through the levees must be provided with gates if the stream will be at high stage for long periods of time. Because of greater development behind a levee, structural failure or overtopping due to underdesign may result in damage greater than if there had been no levee.

Levees and floodwalls were found to be feasible and necessary for protection at Chattanooga in addition to the protection afforded by the multiple-purpose reservoirs. Plans of levees for the protection of Chattanooga to supplement the reservoir protection were prepared and Congress appropriated money to start construction. The project did not materialize, however, because the city did not meet Federal requirements for participation. Levees and channel improvements in conjunction with detention reservoirs also were found to be the most feasible means of protection at Asheville, N. C., and its adjacent agricultural land, and at Bristol, Tennessee-Virginia. To date, however, levees and floodwalls have been utilized only in reservoir adjustment problems but have not been utilized in TVA's flood control program.

Channel improvement

The purpose of channel improvement is to make a stream carry more water than formerly, thus reducing overflow and resulting flood damage for given flows. Channel improvements are often used in conjunction with levees and storage reservoirs. Lowering of the flow line by this method will tend to offset the raising caused by confinement by levees. The improvement in carrying capacity of the stream is accomplished by widening, deepening, realigning, or paving the channel. Often a considerable benefit may be obtained by cleaning the existing channel of bars, debris, and snags. In any channel improvement scheme, however, maintenance work will be required to continue the full effectiveness of the improved waterway area.

The magnitude of the flood problem, especially at Chattanooga and on the Mississippi River, precluded the application of such a method as the improvement of the channel of the Tennessee River. Channel improvement has not been employed in TVA's flood control program.

Protecting individual property

Protection of property on an individual basis may be accomplished by a wall or levee or by relocation to higher ground, either by elevating the existing

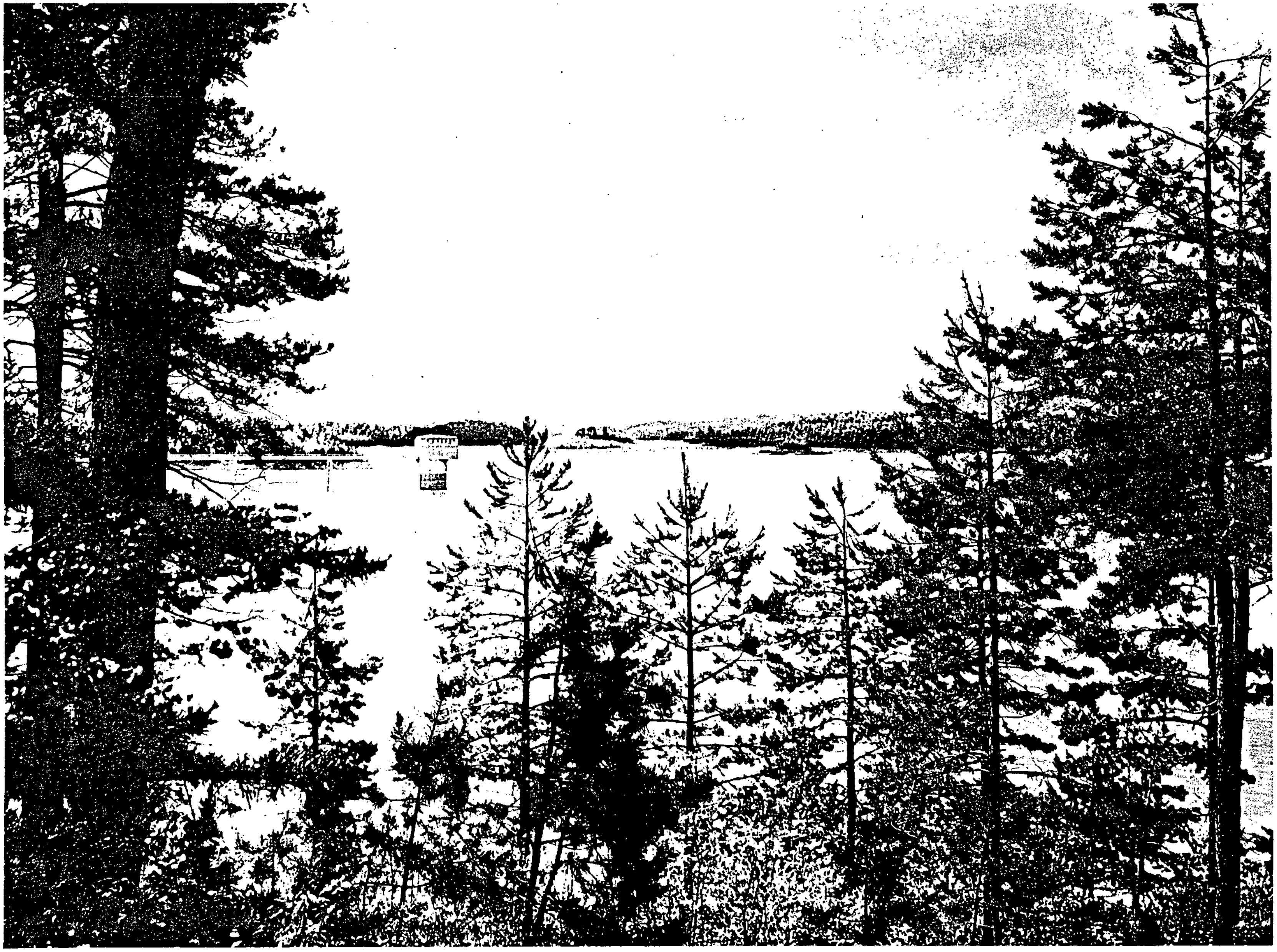


FIGURE 117.—*Nottely Reservoir—stored floodwater reduces downstream flood elevations.*

FIGURE 118.—*Protecting Mississippi levee against wave wash during 1937 high water. (Large waves lashing the earth sides of the levees wash away the earth and are a serious menace to the levees. Here a crew of high water fighters has hastily constructed a board fence backed up by sand bags to protect the levee.)*

site with fill or by removing to a new, higher location. Flood-proofing of individual structures may be accomplished by providing for removable bulkheads for doors and windows, placing check valves on pipelines, sealing walls and floors against seepage, making necessary adjustments to electrical and other utility facilities, and placing protective coatings on equipment. A change in the use of the structure would also fall in this category.

The magnitude of the flood problem, as was the case with channel improvement and small upstream detention basins, precluded the application of individual property protection except in a few relatively insignificant cases.

Flood plain regulations

At new communities or at old communities that are expanding into new areas, controlling development through flood plain regulation is a useful tool in preventing the flood plain or overflow area from becoming occupied with damageable property. The laws or regulations necessary to control the development of such areas must be workable, fair, and practical or their enforcement will be impossible.

LOCAL FLOOD REPORTS

TVA has completed a number of reports to aid communities in the Tennessee Valley in the solution of flood problems which are not taken care of by TVA's reservoir system, and in the practical utilization of lands subject to overflow. These reports are based on studies that TVA has been carrying on in connection with water resources throughout the Tennessee Valley, such as rainfall, runoff, and other technical data with respect to the occurrence and magnitude of floods. A flood history of the stream on which the community is located is included in the report together with an estimate of the maximum probable flood which may reasonably be expected. The reports provide a means of making this information available to states, communities, and other groups interested in local flood problems.

In 1959 TVA submitted to the Congress a report, "A Program for Reducing the National Flood Damage Potential," which was printed by the Senate Committee on Public Works. This report covers TVA's program of local flood damage prevention.

LOCATION OF FLOOD PROBLEMS IN TENNESSEE BASIN

Flood problems exist in the Basin at many cities and towns lying along small streams not affected by the present TVA reservoirs. Some of these problems are serious in their potential damage to life and property as, for example, at Gatlinburg on the West Fork Little Pigeon River, Harriman on the Emory

River, and Asheville on the French Broad River. At these locations serious flooding occurs infrequently, but when the great floods come the damage is severe. At other locations also, flash floods resulting from intense rainfall over small areas cause damage to unsuspecting communities and endanger life.

Table 32 lists the name and location of many of the communities in the Tennessee Basin which are subject to damaging floods. Some of the towns already receive some protection from the multiple-purpose system, but additional works, usually levees or channel improvement, will be required if complete protection is desired against the maximum probable flood. For those towns which are not benefited by the TVA reservoirs a comprehensive study may reveal that reservoirs are necessary, perhaps in combination with levees and channel improvement.

Reports on the feasibility of protection against floods have been prepared by TVA for 22 of the communities listed in table 32 and those for Chattanooga, Harriman, and Asheville have been published. Most of the reports, however, are in memorandum form and have been completed only to the extent necessary to determine whether the costs of flood protection could be justified by the benefits at the time. Reports on flood protection at other communities will be prepared as the need arises. In addition to the 22 communities where TVA has studied the feasibility of flood protection, table 32 lists Lake City, Tenn., where the Corps of Engineers has investigated the flood problem, and Knoxville, Tenn., where channel improvements were constructed by the city of Knoxville and the Knoxville Housing Authority.

Brief descriptions of some of the flood problems and possible methods of reducing the flood damage at the communities listed in table 32 are presented in the following pages. The discussions appear in the order listed in the table except that Dillsboro is included with Sylva, Morgantown with Dayton, and Oakdale with Harriman.

In determining benefits for these projects only those of a tangible nature were used. Consideration of other benefits might have affected the benefit-cost ratio favorably. Costs were based on estimates from sketch plans and necessarily contained large contingency items which might have been substantially lower with better information.

Asheville, North Carolina, and upper French Broad River

Asheville is in Buncombe County in southwestern North Carolina on the French Broad River. A portion of the city known as the Biltmore section is on the Swannanoa River, a tributary of the French Broad. Asheville, the principal city in the upper

TABLE 32.—*Partial list of communities subject to floods in the Tennessee River Basin.*

Community	State	County	Stream	Drainage area, square miles
Asheville[1]	N. C.	Swain	French Broad River	945
Athens	Tenn.	McMinn	Oostanaula Creek	25
Bristol	Tenn. Va.	Sullivan Washington	Beaver Creek	35
Bryson City	N. C.	Swain	Tuskasegee River	655
Canton	N. C.	Haywood	Pigeon River	133
Centerville	Tenn.	Hickman	Duck River	2,048
Chattanooga	Tenn.	Hamilton	Tennessee River	21,400
Cherokee	N. C.	Swain	Oconaluftee River	131
Columbia	Tenn.	Maury	Duck River	1,208
Damascus	Va.	Washington	Laurel Creek Beaverdam Creek	100 56
Dayton	Tenn.	Rhea	Richland Creek	50
Dillsboro	N. C.	Jackson	Tuskasegee River	347
Elizabethton	Tenn.	Carter	Watauga River Doe River	692 137
Fayetteville	Tenn.	Lincoln	Elk River	827
Gatlinburg	Tenn.	Sevier	West Fork Little Pigeon River	133
Harriman	Tenn.	Roane	Emory River	827
Kingsport	Tenn.	Sullivan	South Fork Holston River	1,931
Knoxville	Tenn.	Knox	First, Second, and Third Creeks	22[2]
Lake City	Tenn.	Anderson	Coal Creek	24
Lewisburg	Tenn.	Marshall	Big Rock Creek	25
Morgantown	Tenn.	Rhea	Richland Creek	50
Oakdale	Tenn.	Morgan	Emory River	764
Pulaski	Tenn.	Giles	Richland Creek	366
Roan Mountain	Tenn.	Carter	Doe River	40
Shelbyville	Tenn.	Bedford	Duck River	481
Sweetwater	Tenn.	Monroe	Sweetwater Creek	21
Sylva	N. C.	Jackson	Scott Creek	51

1. Includes upper French Broad River.
2. First Creek only.

French Broad River region, had a population of 60,192 in 1960. Figure 71, page 100, is a map of the French Broad watershed.

As a result of damage caused by the two floods of August 1940, TVA was asked by a committee of residents of the area to make surveys and investigations of methods for controlling floods. A preliminary report was published in August 1942 which presented two alternative plans. The recommended plan included flood control for the agricultural lands lying along the French Broad River upstream from Asheville. Later, in June 1949, a final report was prepared on the basis of the recommendation of the preliminary report. This final report included preliminary plans and estimates of cost of seven detention reservoirs on tributaries and levees along the French Broad River, all based on detailed surveys. There was also included a comparison of annual benefits and costs which showed a ratio of benefits to cost 1.30:1.

An investigation of the upper French Broad flood problem was made by the Corps of Engineers in June 1950, but thus far (early 1961) no report has been issued by the Corps.

The principal flooded area in Asheville lies between the right bank of the French Broad River and the Southern Railway, and along the Swannanoa River. The area is a narrow strip containing many industries. When floods exceed 9 feet on the gage, overflow begins. One of the greatest floods occurred in July 1916 with a peak stage of 23.1 feet, or 14.1 feet above flood stage. Assuming the status of the city as of December 1944, the damage in this flood in Asheville and up the Swannanoa River would have been \$4,812,800. On the basis of 1958 conditions this damage would be in the order of \$9,000,000. Other large floods occurred in August 1928 and twice in August 1940—on the 13th and 30th (fig. 119). Damages in the August 13, 1940, flood, assuming the status of the city as of December 1944, were estimated at \$422,000 or about \$800,000 on the basis of 1958 values. Old records indicate that an early flood probably exceeded that of 1916.

The Asheville flood protection plan presented in the report of June 1949 was based on a maximum probable flood of 154,000 cubic feet per second, which is 40 percent greater than the flood of 1916. Computations showed that the flow of 154,000 cubic

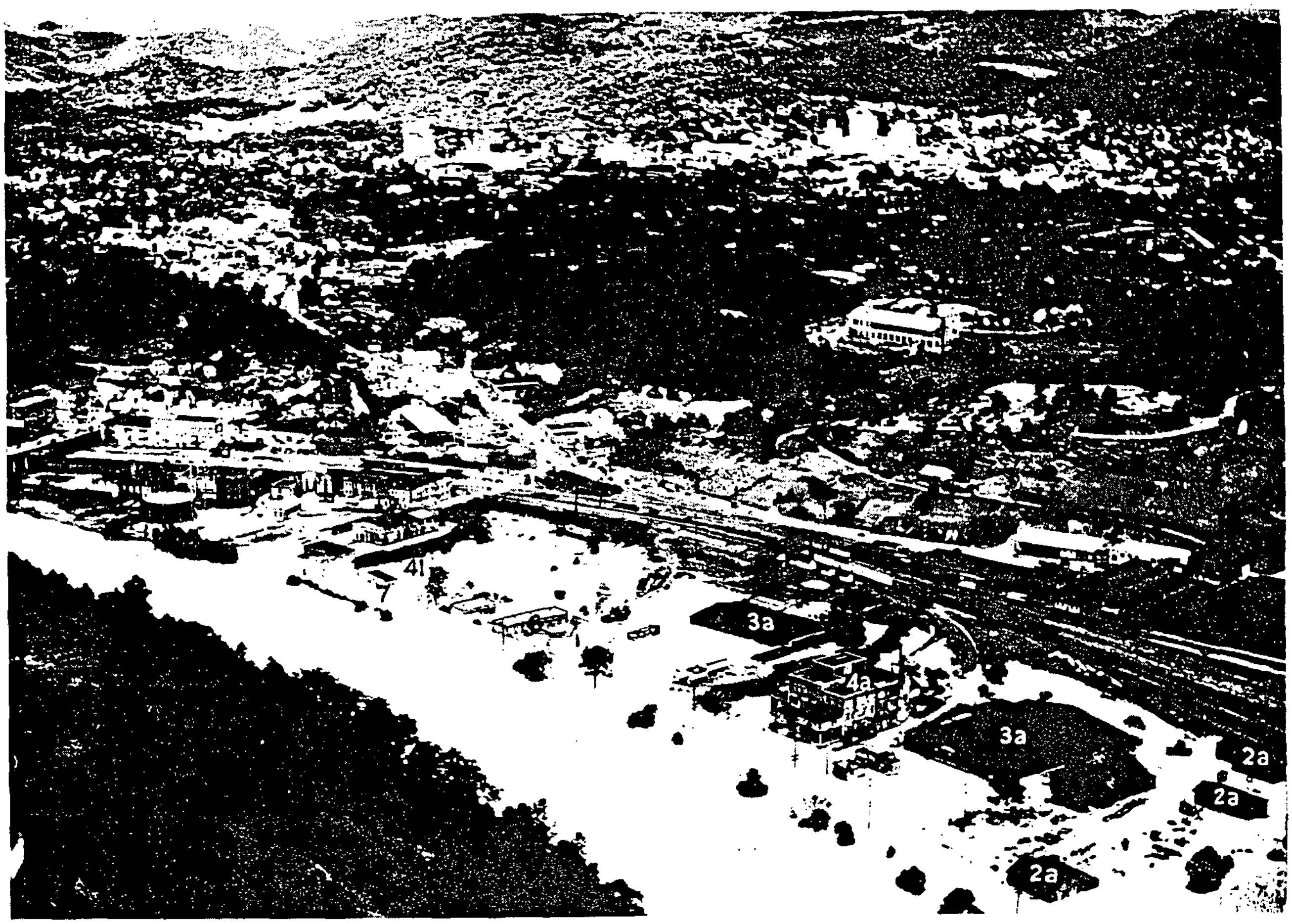


FIGURE 119.—*Asheville waterfront on August 30, 1940 (photo by Asheville Citizen).*

feet per second could be reduced to 96,000 cubic feet per second by the seven detention reservoirs proposed on the upstream tributaries. Levees to a height corresponding to 96,000 cubic feet per second, with an appropriate freeboard, were therefore necessary to confine the 96,000 cubic feet per second flow to the stream channel. The plan also included improvement of the channel and construction of levees for about 17 miles in the agricultural region upstream from Asheville. The height of these levees would be based on the flood of August 13, 1940, as regulated by the detention reservoirs. The proposed detention basins and levees would reduce the flooding on approximately 12,000 acres of farm land.

Athens, Tennessee

Athens, county seat of McMinn County, Tennessee, is a town of 12,103 people (1960) located to the north and west of Oostanaula Creek, a tributary of the Hiwassee River. About two blocks of the business district close to a loop of the creek lie at a low elevation and are subject to flooding. In the flood of January 1946, the town suffered a total damage of $18,600. Figure 67, page 86, shows the location of Athens in the Hiwassee River Basin.

A detention reservoir above the town, while possible, would cost far more than can be justified by the damage. A low levee about two blocks long, however, together with the provision of gates for the sewers and drains entering the creek in this area, would give adequate protection and might be built at a low enough cost to be justified. A detailed survey would be necessary to determine if this is so.

Bristol, Tennessee-Virginia

The adjoining cities of Bristol, Tennessee, and Bristol, Virginia, are located on Beaver Creek, a tributary of the South Fork Holston River. The center line of the main street of the two cities is the dividing line between the two states. Bristol, Tennessee, is in Sullivan County, and Bristol, Virginia, is in Washington County, and figure 70, page 96, shows their location in the Holston River Basin. The combined population in 1960 was 34,726, with 17,582 in Tennessee and 17,144 in Virginia.

Beaver Creek flows generally in a southwesterly direction, joining the South Fork Holston River downstream from South Holston Dam and Bluff City. The principal tributaries entering Beaver Creek above Bristol are Goose Creek and Clear Creek. Beaver Creek flows through the central business district, and a covered portion of the creek, completed in 1925, forms a main highway for several blocks. The natural carrying capacity of the stream is greatly restricted by the covered portion and by many buildings, bridges, and other encroachments. The drainage area of Beaver Creek at the state line is about 35 square miles.

The flood problem at Bristol has been recognized for many years. As early as 1870 improvements to the channel were attempted, and as the city increased in population the demands for the control of floods also increased. Following the flood of July 1, 1929, the District Engineer of the Corps of Engineers made a report on October 10, 1929, which recommended that the cities make a survey of the problem and, in line with Federal policy at that time, that the execution of the plans be carried out by the cities.

A report dated August 5, 1930, was prepared jointly by the city engineers of the two cities. It recommended the deepening and widening the stream channel, building flood walls, raising bridges, and installing flood gates on sewer outlets, all on the assumption of a flow of 3,600 cubic feet per second at the Norfolk and Western Railway culvert and at a combined cost to both cities of $75,000. A later report dated September 1, 1933, dealt with improvements to the stream channel downstream from State Street in Tennessee, making about the same recommendations as were made in the report of August 5, 1930. Some of the recommended improvements were made with the help of the Works Progress Administration, but the work was not completed.

The highest known flood of March 1867 reached an elevation of 1674 at the Moore Street entrance to the covered portion. The flood of July 1929 was only 1.3 feet lower, reaching elevation 1672.7. Other high floods occurred in 1879, 1875, 1905, and 1917. There were five floods in 1923 which exceeded the damaging stage of 1668. At this elevation flooding begins by backing through the sewers. The record shows that floods occur on an average of once eyery 3 or 4 years. Other damaging floods might have occurred during this period without any record of their height being made, since no stream gage has been maintained here.

Several reservoir sites are available upstream from Bristol, and if these were developed as detention basins with an outlet capacity equivalent to the capacity of Beaver Creek through Bristol, the danger from floods would be eliminated. Because the flood plain in the city is thickly built up, there seems to be no possibility of levees or walls to confine the flow. There is a possibility of a relief channel or tunnel to divert some of the flow of Beaver Creek past the built up area. A flood protection report prepared in 1953 considered three plans for reducing flood damage. One of the plans, consisting of two detention reservoirs, would reduce all floods—equivalent to those recorded— except that of 1867 to a stage where there would be no significant damage. Preliminary analysis indicated that if this plan were carried out, the average annual benefits would be somewhat greater than the annual charges it would incur.

Bryson City, North Carolina

Bryson City, county seat of Swain County, North Carolina, is a town of 1,084 people (1960) located on both sides of the Tuckasegee River, at the upper limit of backwater from Fontana Reservoir. It is shown on the map of the Little Tennessee River Basin, figure 69, page 92. In the flood of August 30, 1940, the river water flowed several feet deep over a large part of the town (fig. 116, page 184), and caused damage estimated at $36,000. With 1958 values this damage would be about $100,000. There is a long record which shows floods of seriously damaging height in 1840, 1867, 1886, 1902, 1906, and 1940.

Bryson City could be protected by two lines of levees, one on each side of the river, having a total length of about 6,500 feet. The levees would require the removal of a number of stores and houses, and the raising of the bridge across the river. Rough estimates indicated that the average annual cost of levees would be several times the annual damage, and levee protection was not considered economically feasible. Neither was protection by detention reservoirs considered economical.

Canton, North Carolina

Canton, an industrial town with a 1960 population of 5,068, is situated on both sides of the Pigeon River in Haywood County, North Carolina, in the watershed of the French Broad River (fig. 71, page 100). In the two floods of August 1940, Canton suffered considerable damage; in the higher of the two the damage was almost $200,000, about one-third of what it would be with 1958 values. Two-thirds of this damage was inflicted on the plant and yards of the Champion Paper and Fibre Company, which occupies the northern or downstream part of the town. There were other damaging floods in 1876, 1893, 1902, 1928, and 1949.

The Champion Paper and Fibre Company has built protective works which would prevent serious overflow in the plant area in floods as high as the maximum so far experienced.

It would be possible to protect the city by levees running along both banks of the river, with a total length of about two and one-half miles. They would require the removal of some buildings, the raising of both highway bridges, the provision of gates at the two railroad bridges, and the loss of trackage in the pulpwood storage yards. Estimates indicated, however, that the annual cost of the levees would be greater than the annual benefits and, on a cost basis, construction was not considered justified; detention reservoirs came even further from economic justification.

Centerville, Tennessee

Centerville, on Duck River (fig. 64, page 80), is the county seat of Hickman County and has a population of 1,678 (1960). The town was not seriously damaged by the record flood of 1948 because most of it is located on a hill above the river. Although the flood crest exceeded the overflow stage of 22 feet by almost 16 feet, only a small amount of damage was suffered. The principal damage resulted from flooding of the source of the city's water supply and the pumping station. All but one of the access highways were flooded. Indirect damage resulted from the closing of the largest industry for a short time.

It seemed doubtful that flood control works could be justified for Centerville on the basis of damages incurred in past floods. An upstream storage reservoir built primarily for flood control at other points would aid in protecting Centerville, and benefits to that city could be used to help justify the costs of the project.

Chattanooga, Tennessee

The city of Chattanooga, with a population of 130,009 in 1960, is located in Hamilton County on the Tennessee River just above the mountain ridge which separates the Basin into distinct upper and lower portions. Chattanooga's location in the Valley with respect to the major tributary basins is shown in figure 63, page 79. Before the completion of the TVA reservoir system, the flood damage here was the most serious in the Tennessee Basin, amounting to 90 percent of the average annual flood damages within the portion of the Basin now subject to regulation by the present reservoir system.

The city is the principal trading center for an area of 100 to 150 miles in diameter and is a leading industrial city in the Tennessee Valley area, being favorably located to sources of raw materials for its industries. Important highways and railways enter the city carrying a large amount of through-traffic as well as a substantial amount originating or terminating within the city.

The highest known flood occurred in March 1867. It reached a stage of 57.9 feet, or 27.9 feet above flood stage, and covered an area of approximately 8,900 acres within the city limits. Most of the area that was under water in Chattanooga and vicinity during this flood is shown in figure 120. It is estimated that the equivalent of this flood could be reduced by operation of the existing reservoirs to a stage between 40 and 44 feet, with a corresponding area flooded of as much as 5,700 acres. At flood stage of 30 feet there are 1,600 acres flooded. The maximum probable flood which was used as the basis for determining the height of proposed levees would reach a stage of 77.3 feet under natural conditions and 60 feet after regulation by the reservoirs. The yearly and seasonal distribution of floods at Chattanooga is shown in figure 25, page 22.

Surveys of potential flood damage in Chattanooga were made by TVA in 1938 and 1948, and a supplemental survey, principally from aerial photographs, was made in 1953. The determination of flood damages is given in greater detail in chapter 12. Average annual damages, assuming the occurrence of all floods, both with and without regulation, and assuming the status of the city as of 1938, 1948, and 1953, are as follows:

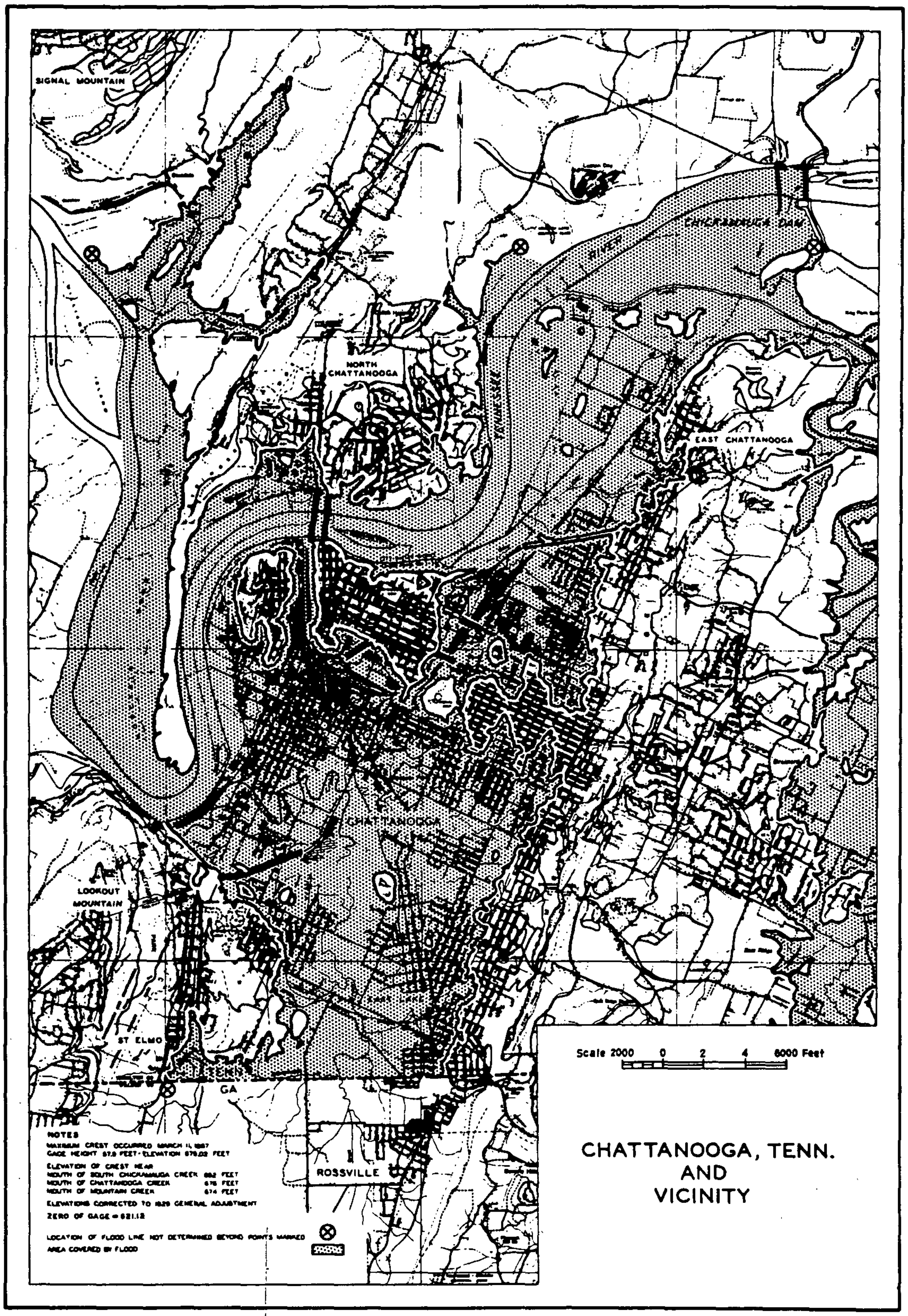


FIGURE 120.—*Area of Chattanooga and vicinity covered by flood of 1867.*

	Status of Chattanooga as of		
	1938	1948	1953
Annual average damage, unregulated	$1,893,000	$5,323,000	$5,483,000
Annual average damage, regulated	75,000	229,000	231,000
Annual damage prevented	$1,818,000	$5,094,000	$5,252,000

The estimated damage—as of 1953—in the regulated maximum probable flood (stage 60 feet) is more than $100,000,000 and in the regulated maximum known flood is $12,500,000.

With the TVA reservoir system, about 96 percent of the average annual damage at Chattanooga will be prevented, but the remaining 4 percent ($231,000) is a substantial amount. Moreover, the increased development of the low land still subject to flooding is rapidly creating new values to suffer flood damage. The prevention of the flooding of this low land by levees is a necessary part of flood protection to the city as envisaged in reports by TVA and the Corps of Engineers.

In 1939 a report entitled "The Chattanooga Flood Control Problem" was prepared by TVA and published as House Document No. 91, 76th Congress, 1st Session. Later in the same year a report by the Corps of Engineers was published as House Document No. 479, 76th Congress, 2d Session. These reports describe in detail the city and the flood problem, the basis for the adopted design flood, and the degree of control to be provided by reservoir storage and local protective works.

The TVA report concluded that both reservoir control and levees were required for complete protection against a maximum probable flood, which was 60 percent greater than the maximum known flood. The recommended plan included 4,000,000 acre-feet of storage on the five principal tributaries, in addition to that in the main Tennessee River projects, and a system of levees and walls to protect against a river stage of 60 feet on the Chattanooga gage corresponding to a discharge of 486,000 cubic feet per second. Even if the tributary reservoirs could completely control their drainage area of 13,420 square miles, the maximum probable flood from the remaining 7,980 square miles immediately above Chattanooga still could be about 450,000 cubic feet per second.

TVA has built ten reservoirs on tributary streams having a total storage reservation for flood control on March 15 of 4,600,000 acre-feet, and a small degree of control has been provided in three main Tennessee River reservoirs. Three of the ten tributary reservoirs—Watauga, South Holston, and Boone—although not directly effective on the crest of the Chattanooga flood, aid materially in the operation of Cherokee Reservoir during extreme floods. Even excluding storage in these three reservoirs, the flood control reservation on March 15 exceeds the 4,000,000 acre-feet recommended in the TVA report.

The levee system proposed by the Corps of Engineers was authorized by Congress in 1941, and in 1946 the sum of $500,000 was appropriated to start construction, subject to the compliance by the city with certain requirements of the Flood Control Act of 1936 as amended. This authorization expired in December 1953, because these requirements were not met.

TVA has repeatedly informed the city of the possibility of occurrence of a flood greater than that of 1867, and has urged it to take the necessary steps to ensure the construction of the protective works to a height corresponding to 60 feet on the gage.

Cherokee, North Carolina

Cherokee is a small but growing tourist village in Swain County, North Carolina. It is at the edge of the Great Smoky Mountains National Park and within the Cherokee Indian Reservation. Its location is shown on the map of the Little Tennessee River watershed (fig. 69, page 92). The Oconaluftee River, flowing southward from the mountains, passes through the village, which is scattered along both banks for a considerable distance. A flood in July 1955 caused about $6,000 in damage, over two-thirds of which was accounted for by the washout or damage of a number of bridges. There were higher floods in 1936, 1946, and 1947, but there is no estimate of the amount of damage from these floods.

The difficulty in the way of protection for Cherokee is the scattered nature of the damage. Levees or walls would not protect the bridges, and would need to be very long to protect all the low-lying buildings, so they would undoubtedly cost more than the small amount which could be justified for this purpose. A detention reservoir could be made large enough to give complete protection, but it would cost far more than the small amount of damage would justify.

Columbia, Tennessee

Columbia, county seat of Maury County, Tennessee, is the largest city in the Duck River Basin (fig. 64, page 80); its population in 1960 was 17,624. There are buildings on the flood plain on both sides of the Duck River which are subject to inundation from high floods.

The three greatest known floods at Columbia were those of February 1948 (fig. 52, page 56), March 1902, and March 1955. The February 1948 flood crest was a maximum at Columbia, exceeding the March 1902 crest by 3.8 feet and the March 1955 peak by 7.0 feet. The 1929 flood, which was second highest at Shelbyville, ranked fifth at Columbia.

The city suffered heavily from the 1948 overflow. The city water supply was cut off from February 13 to February 18. One of the two substations serving the city was flooded, and the power supply was reduced to one-half for several days. Seven industries were affected by interruptions to power and water service in addition to four that were flooded. (The water plant has since been rebuilt at a flood-free location.) Damage occurred to 102 homes of which 42 were in the Riverside community. In a large group of the Riverside homes, which were known to be above the 1902 flood level, no attempt was made to move furniture until it was too late. Twenty business places were flooded, and four were affected by the shortage of power and water. Flooding of these stores ranged in depth from 5 feet to 14 feet.

The total estimated flood loss in Columbia as a result of the February 1948 flood was \$213,000. No estimate is available for the damage in the earlier floods, but newspaper accounts indicate that the same general areas were affected.

A report on flood protection issued in 1953 outlined a system of protective levees, one about 4,600 feet long along the right bank to protect the residences there, and a shorter one across a creek valley on the left bank to protect the water plant, a large warehouse, and other buildings. Because average annual benefits were estimated to be relatively small compared with the average annual cost, it was concluded that levee construction, insofar as costs were concerned, could not be justified at that time.

Damascus, Virginia

Damascus is a small industrial town (1960 population 1,485) in the southeastern corner of Washington County, Virginia, close to the northeastern corner of Tennessee and on the edge of the Jefferson National Forest. It lies at the junction of Laurel Creek and Beaverdam Creek, both of which emerge from the heavily wooded mountain country, and is shown on the map of the Holston River Basin (fig. 70, page 96). The business district is on a low-lying peninsula between these two streams. A considerable portion of Damascus would be overflowed by a recurrence of a flood equivalent to that of May 1901, the highest known at Damascus. Other floods in 1940, 1955, 1956, and 1957 also caused damage.

If flood protection were to be provided at Damascus, it could probably be best accomplished by a levee around the low part of the town, but it appears that a great length would be needed for the few blocks protected, so the cost would be high. There is no possibility of detention reservoirs, because of the size of the streams and the presence of roads and railroads along them.

Dayton and Morgantown, Tennessee

Dayton, having a population in 1960 of 3,500, is the county seat of Rhea County, Tennessee, and is located on Richland Creek where it discharges into Chickamauga Reservoir on the west side, about 4 miles upstream from its original junction with the Tennessee River. It is 35 miles northeast of Chattanooga. Although Dayton is across the reservoir from the Hiwassee River watershed it is shown in figure 67, page 86, which is a map of that watershed. Morgantown, about one-fourth the size of Dayton, is located about 1 mile farther upstream on Richland Creek. Above the two towns the creek drains a high plateau by means of a fan-shaped system of tributaries. These have cut deeply into the plateau, leaving steep streambeds and hillsides conducive to a rapid concentration of runoff resulting in flash floods which can rise and sweep through the towns in a matter of hours. Below Dayton the creek flows in a comparatively narrow gorge. The drainage area above Dayton is 52.3 square miles, and above Morgantown is 50.2 square miles.

Dayton and Morgantown lie on a wide, flat, and low flood plain which is completely different in character from the areas above and below. Business buildings, small industries, and residences are subject to periodic flooding on this flood plain. Industries within areas likely to be flooded include underwear and hosiery mills, a bottling plant, and a basket factory. One large food-freezing plant and other industries are outside the limits of direct flood damage. In Dayton, U. S. Highway No. 27 follows the main street of the town, Market Street, and is covered whenever flooding of any importance takes place. A severe flood would cause an interruption of traffic and possible failure of the existing highway bridge over the creek. The main line of the Southern Railway passes through Dayton at its upstream edge, but only a flood of extreme proportions could interfere with the movement of rail traffic.

In the past, Dayton has been flooded more severely by backwater from the Tennessee River than by headwater floods of Richland Creek. The danger from the Tennessee River, however, has been largely eliminated by the TVA reservoir system. The maximum known flood of 1867 could have been regulated to elevation 690, the elevation to which land was purchased for Chickamauga Reservoir, but the danger from headwater floods on Richland Creek remains. During the 27 years from 1927 to 1953, Richland Creek floods exceeded a damaging stage, elevation 692, eight times. The highest known flood, February 27, 1903, reached elevation 695.

Local protection in Dayton by means of levees, widening and deepening of the stream channel, and lengthening and raising of the highway bridge, would be feasible but would require grade adjustments of the highway and some streets, adversely affecting adjacent property.

Flood protection for the smaller town of Morgantown was not investigated at the time of the Dayton studies.

Elizabethton, Tennessee

Elizabethton is in northeastern Tennessee on the left bank of the Watauga River, 10.5 miles downstream from Watauga Dam and 40 miles upstream from Kingsport. It is shown on the map of the

Holston River Basin (fig. 70, page 96). Its population was 10,896 in 1960. As county seat of Carter County, it serves a large surrounding area. Two large rayon plants and other smaller plants are located in the city, providing employment for a large number of workers.

Most of the city lies outside the flooded area, but many houses have been built on the low ground lying within a strip about one-half mile wide along the river. A developed area along the Doe River, a large tributary of the Watauga River, is also affected by floods on that stream.

The drainage area of the Watauga River above Elizabethton is 692 square miles, of which 137 square miles is drained by the Doe River. The record of floods at Elizabethton shows that the highest occurred in May 1901, reaching a crest stage of 21.5 feet on the gage. The second highest was that of August 13, 1940, which reached a stage of 20.87 feet. Other floods occurred in February 1902, July 1916, March 1935, and December 1942. Historic flood records at Butler, 20 miles upstream on the Watauga River, before creation of Watauga Reservoir, indicate that still other large floods occurred in March 1867, February 1875, March 1886, April 1896, February 1897, October 1900, and December 1901. Except for those of 1935 and 1942, all these floods exceeded flood stage of 14 feet. The duration of these floods was usually short, lasting less than two days. The flood of August 1940 crested two hours after flood stage was reached and was above that stage for nine hours; within the city limits the area flooded was approximately 470 acres.

The contribution of the Doe River to the 1940 flood was almost negligible, the storm being located over the portion of the basin above the Watauga dam site. This reservoir in such a flood could be highly effective in reducing flood stages at Elizabethton. On the other hand, the Doe River contribution in the May 1901 flood was a substantial amount, and would produce high stages at Elizabethton regardless of the storage in Watauga Reservoir.

Following the 1940 flood, a survey revealed damages of $276,000 at Elizabethton, of which $212,000 was attributed to the shut-down of the rayon plants for several days because of flooding of the filter plant, and $29,000 was for loss or partial damage to 130 houses in the city. The community of Rio Vista, on the Watauga River 3 miles downstream from Elizabethton, also suffered heavy damage in August 1940.

Watauga Reservoir is on the Watauga River about 14 miles upstream from Elizabethton. Its flood storage reservation on January 1 is 256,200 acre-feet, equivalent to 10.26 inches over the drainage area. On March 15, the reservation is 152,900 acre-feet, or 6.13 inches, and the reservation for flood storage is never less than 109,000 acre-feet, or 4.37 inches. Additional uncontrolled storage would also be effective in reducing the flood even though the water level rose above the spillway crest.

These storage reservations in Watauga Reservoir provide a high degree of control for the 468-square-mile drainage area above the dam. This area is 67.5 percent of the area above Elizabethton. Studies of operation of Watauga Reservoir show the flood of August 1940 could have been reduced well below the damaging stage. Although information for the May 1901 flood is limited, particularly with respect to the flow in the Doe River, computations show that it could have been reduced to a stage of 16 feet, or 2 feet above flood stage.

The unregulated maximum probable flood at Elizabethton has been estimated to be 158,000 cubic feet per second, equivalent to 6,000 times the square root of the drainage area. This flow would produce a stage of about 30 feet, or 16 feet above flood stage. With Watauga Reservoir in operation, this flood would be reduced substantially, but the large flow contribution from the area below Watauga Dam, including the Doe River, would still produce a flood about equal to that of the May 1901 flood (fig. 149, page 276). This would be equivalent to 23 feet on the gage, or 9 feet above flood stage.

In a flood protection report prepared in 1956, several plans for protection against damage that could result after the construction of Watauga Dam were investigated. In addition to Elizabethton the study included Rio Vista on the Watauga River and Valley Forge and Hampton on the Doe River. The latter two towns were included because consideration was given to flood protections by means of reservoirs on the Doe River. Estimates indicated, however, that protection by levees and walls, or by flood storage reservoirs on the Doe River, or by a combination of the two methods could not, on a cost basis, be justified at that time.

Fayetteville, Tennessee

Fayetteville, county seat of Lincoln County, Tennessee, a city of 6,804 people (1960), is located on Norris Creek at the point where the creek enters Elk River (fig. 65, page 82). A part of the business and residential area is spread along the flood plain of Norris Creek and is subject to overflow from floods on Elk River.

A number of large floods on Elk River have inundated the low area of Fayetteville. The greatest, indicated by a high water mark chiseled on a rock bluff below the mouth of Norris Creek, occurred in 1842. A flood in March 1929 came within a few inches of reaching the same mark. The recent flood in January 1949 was 0.2 foot below the 1929 crest at the stream gage above town but was said to have been 5 inches higher in Fayetteville. These three floods were one-half foot or more higher than any others in the experience of the town, but it is doubtful if they represent the maximum that might be expected.

A survey of damages resulting from the near-maximum flood of January 1949 showed a total loss, including intangible damages, amounting to approximately \$83,000. Sixty percent of the loss was to Fayetteville business places along Norris Creek, while damages to industries and residential property ranged between 15 and 20 percent each. The flood damage in February 1948, for a flood a foot or more lower, was \$43,000. Here again the commercial loss was approximately half the total, and industry and homeowners shared most of the balance.

No flood control works have been built either on Norris Creek or on Elk River above Fayetteville. However, Woods Reservoir, which provides a water supply for the Arnold Engineering Development Center, is operated to reduce the level of floods below its location near Estill Springs. The crest of the February 1957 flood, the fourth highest known at Fayetteville, was lowered somewhat by operation of this reservoir.

A report issued in 1953 shows that most of the damage in Fayetteville could be prevented by a levee about 4,000 feet long along the bank of Norris Creek. The drainage basin behind this proposed levee is comparatively large, so that a pumping plant would be required in conjunction with the levee. Rough estimates indicated, however, that such a protective system could not be economically justified at that time.

Gatlinburg, Tennessee

Gatlinburg (1960 population 1,764) lies at the foot of the Great Smoky Mountains in Sevier County, Tennessee, on the West Fork of the Little Pigeon River. It is shown on the map of the French Broad River watershed (fig. 71, page 100). Because of its location near the Great Smoky Mountains National Park it has become a tourist center with numerous curio and native craftwork shops, tourist motels and cabins, and hotels. In recent years the main street has been widened and paved, but without any substantial change in grade with respect to that of the stream. Many of the tourist cabins are built close to the banks of the stream and some are on an island, which is flooded with only a moderate increase in stage. Much of the channel width has been restricted by walls in recent years, and the stream channels now have only a fraction of their original capacity.

The basin of the West Fork of the Little Pigeon River is extremely mountainous with elevations ranging from nearly 1,300 feet to above 6,000 feet. The tributary streams, therefore, have steep slopes. Several of them join the main stream in the town, thus indicating the possibility of a heavy flow concentration. The basin is almost entirely in forest.

Records of extreme rainfall and flood heights are limited because of the absence of gages. A storm in July 1942 resulted in a rise of 4 or 5 feet, which was reported to be the highest flood in the preceding 14 years. An island on which there were several cabins was partially flooded. The damage, however, was small, and the main section of the town was not affected. At the time of this flood an old resident stated that he remembered a flood which he believed would have flooded most of the present town of Gatlinburg.

The numerous encroachments on the natural flood waterway of the river have set up a condition which is particularly hazardous to life and property. An intense rainstorm on the 31.8 square miles of mountainous watershed above the city could cause a great flood that would strike the city with little or no warning. On the afternoon of September 1, 1951, a heavy rain of 4 inches or more occurred in the vicinity of Mt. LeConte and Clingmans Dome, two of the highest peaks of the Great Smoky Mountains. Less than an inch fell at Gatlinburg. In only a few minutes after the end of the brief storm, the flood was in Gatlinburg where the river rose 10 feet in 15 minutes and spread out to a maximum width of 600 feet. Tangible damages in the city were estimated at \$42,000.

The largest known flood at Gatlinburg occurred on April 1, 1896. The level of this flood was 4 or 5 feet higher than that of 1951. The 1896 discharge with the present development in the flood plain would produce a flood considerably higher than the stage that actually occurred and would cause tremendous damage and probably loss of life if the flood came at night. Experience on similar drainage basins in the Gatlinburg region indicates that even larger floods than that of 1896 are possible.

Many of the occupants of the cabins and shops may be unfamiliar with the fact that the stream can rise suddenly and thus make it difficult for them to get to high ground. An example of a flood which rose rapidly was that of August 1947. Although there was no damage at Gatlinburg in this flood, one person was drowned some distance upstream where a "wall of water" descended on a group of people near the Chimneys, a well-known peak of the Great Smokies. The storm which produced the flood lasted only for an hour. The rainfall amounted to over 3 inches at Newfound Gap (elevation 5,000 feet), but there was no rain below elevation 3,000.

The possibility of constructing levees or flood walls to protect the portion of the town subject to flooding seems limited because of the development close to the stream banks. It may be possible to locate a detention basin site which would provide sufficient control, but investigations have not yet been completed.

The situation at Gatlinburg is critical with respect to hazard to both life and property. A flood similar to those experienced in recent years on nearby small streams would destroy all small buildings that are on low ground near the stream channel. Along the valley of such streams as Baskins Creek there is also danger from slides of saturated overburden on the mountain slopes, such as occurred above town in the 1951 flood. The rise of the flood waters would be so rapid that access to safety on high ground would be quickly cut off. Such a flood could occur any time of year, when the town is crowded with summer visitors or during the winter. It could rise and fall during a single night while residents of cabins were asleep.

Harriman and Oakdale, Tennessee

The Emory River drains 865 square miles by means of a fan-shaped system of tributaries originating on the Cumberland Plateau. The topography, character of the soil, and shape of the area combine to cause efficient concentration and high runoff. The Emory River is in the Clinch River watershed at its southwest end and is shown in figure 68, page 88.

The city of Harriman, population 5,931 in 1960, occupies practically all of a relatively flat area on the left bank within a large bend in the Emory River, which borders the city on three sides. This bend is roughly a mile in diameter. Oakdale, about 6 miles upstream from Harriman, is located partly within a smaller bend on the right bank and partly along the left bank above this bend. Oakdale had a population of 470 in 1960.

The March 1929 flood outranks all others in the Emory River Basin, and in peak rate per square mile of drainage area it ranks among the highest in the Tennessee River Basin as well. It was caused by a storm traveling northeast from the Gulf in which rainfall of from 6 to 11 inches fell in from 12 to 15 hours, probably one-half of which fell in a 2-hour period. The resulting flood caused great damage in the Emory River Basin, particularly at Oakdale and Harriman (fig. 121).

The protection of Harriman by means of upstream reservoirs, by levees and flood walls, and by a combination of both, has been considered in past studies, which have shown that the cost of protection would exceed the benefits provided. A report prepared in June 1939 concluded that the cost of a project which would afford a suitable degree of flood control was not justified at that time. These studies were made, however, without the consideration of an unexpected lowering of flood heights in Harriman due to improved channel conditions resulting from the construction of Watts Bar Reservoir, completed in 1942.

If credit were taken for this important reduction in flood heights due to Watts Bar Reservoir—amounting to about 7 feet in a large flood—in reconsidering flood protection for Harriman, the comparison of cost of protection to expected benefits should be more favorable. A new appraisal including consideration of multiple-purpose reservoirs should be made to determine the present economic feasibility of flood protection for Harriman. In the event that one or more storage reservoirs might become a part of the most feasible protection plan for Harriman, then these same reservoirs also would provide some protection for Oakdale.

Kingsport, Tennessee

Kingsport is in Sullivan County, Tennessee, on the South Fork Holston River. It is 51 miles downstream from Watauga Dam, 44 miles downstream from South Holston Dam, 13 miles downstream from Boone Dam, and 2½ miles downstream from Fort

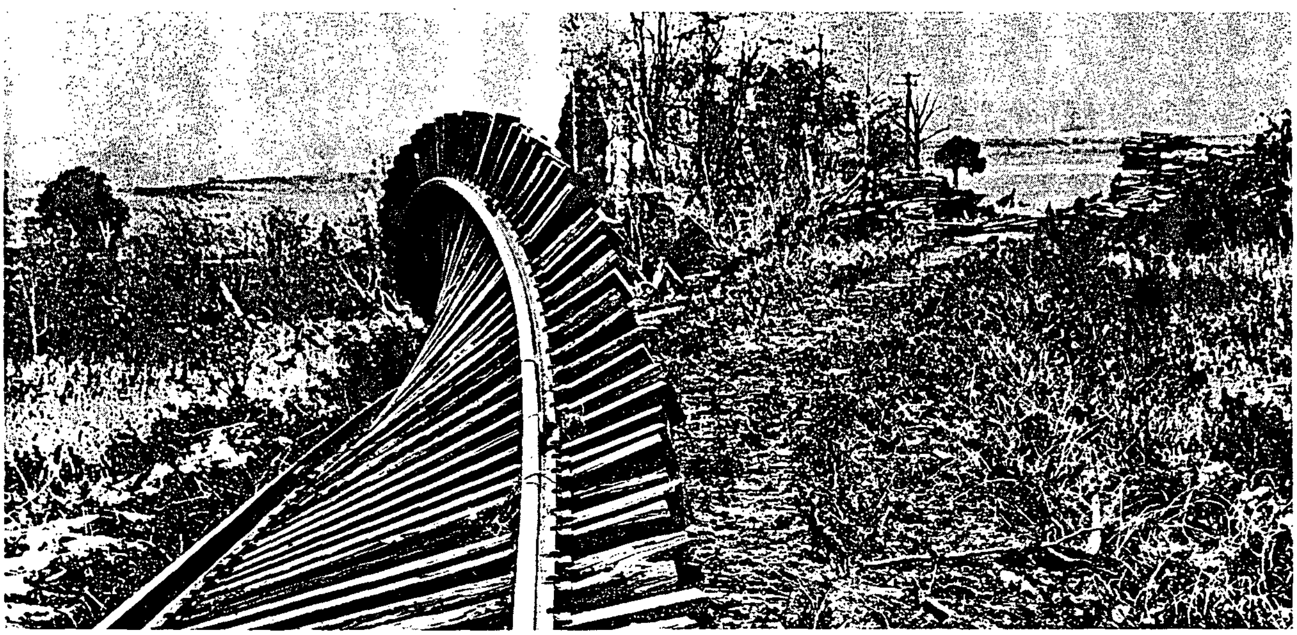


FIGURE 121.—*Destruction wrought to Harriman and Northeastern Railroad tracks near Harriman by March 1929 Emory River flood.*

Patrick Henry Dam. It lies a short distance upstream from the junction of the North and South Forks of the Holston River, and is shown on the map of the Holston River Basin, (fig. 70, page 96). The population of Kingsport was 26,314 in 1960.

Although most of the city lies well above the area subject to flooding, several industrial plants would be damaged by a repetition of the maximum known flood. Long Island, the upper end of which is developed with many houses, was almost completely covered in the smaller flood of August 1940.

The highest flood at Kingsport occurred in May 1901 with a crest stage of 23 feet on the gage at mile 5.67 then in use. The flood of March 1867 was practically the same height. The flood of August 1950 (stage 18.8 feet) was seventh in order of magnitude. Flood stage is about 14 feet. A stream gage, established in 1925, was located a short distance upstream from the head of Long Island until 1953, when it was moved about 2 miles downstream. Flood records at these gages are supplemented by 15 earlier floods, data for which were obtained from newspaper accounts and interviews with residents. The earliest of these floods occurred in March 1791. The yearly and seasonal distribution of floods at Kingsport is shown in figure 73, page 110.

In the 1940 flood, 126 houses on Long Island and 50 houses in other sections of Kingsport were flooded. The depth and duration of flooding in the houses were not enough to prevent their being occupied the next day. The total damage in Kingsport was estimated to be $43,000, but if the flood had been only 1 foot higher, damage to the Tennessee Eastman Corporation alone would have been about $100,000.

In addition to flooding caused by the South Fork Holston River, a portion of Kingsport is affected by floods on Reedy Creek. This tributary, which enters the South Fork near the lower end of Long Island, has a drainage area of 60 square miles. Floods occurred on Reedy Creek in 1927, 1944, and 1955, but that of 1927 was the greatest known. In the lower reaches of Reedy Creek, elevations of the 1927 flood, if it were to recur, would be substantially higher because of filling which has been made since 1927 on the overflow area.

The drainage area of the South Fork Holston River at the Kingsport stream gage is 1,931 square miles, of which 468 square miles is above Watauga Dam and 703 square miles is above South Holston Dam. These two multiple-purpose projects have relatively large flood-storage reservations, as shown in the following tabulation:

	Flood storage reservation in			
	Watauga		South Holston	
	Acre-feet	Inches[1]	Acre-feet	Inches[1]
January 1	256,200	10.26	286,300	7.64
March 15	155,400	6.23	218,200	5.82
Minimum	109,000	4.37	105,800	2.83

1. Equivalent depth over drainage area above dam.

These storage reservations represent a high degree of control of the combined area of 1,171 square miles above the dams. The effect of these reservoirs on stages at Kingsport (fig. 148, page 276) would be to lower the crest of the maximum known flood of 1901 about to flood stage. The uncontrolled area of 760 square miles, however, is still capable of producing a large flood, and if protection against the maximum probable flood is desired, then additional protective works must be built.

It has been computed that Watauga and South Holston Reservoirs could reduce the maximum probable flood at Kingsport from a stage of 34.2 feet to 28.3 feet. Levees or flood walls to a height corresponding to this stage, therefore, will be necessary to give full protection against floods. The flood control storage reservation in the Boone project during the winter and early spring supplements the control afforded by Watauga and South Holston, and will give additional stage reduction at Kingsport in some floods. However, the maximum probable flood may occur when Boone Reservoir is filled, and no additional crest reduction would be obtained in such a flood. The Fort Patrick Henry project has no flood control storage reservation.

The Boone and Fort Patrick Henry projects, which were completed in 1953, are located on the South Fork Holston River between Kingsport and the South Holston and Watauga Dams. Studies made during the planning of Boone and Fort Patrick Henry indicated that it would be cheaper to provide additional protection from the Kingsport maximum probable flood by means of local protection works than by means of reservoir storage in the Boone project. Studies made in 1956, however, concluded that insofar as costs were concerned no system of additional or supplemental flood protection by means of levees and flood walls could be justified at that time.

Knoxville, Tennessee

Knoxville is fairly well protected by flood control storage on the French Broad and Holston Rivers from floods on the Tennessee River as great as those of the past 167 years. The city (1960 population 111,827) is on the Tennessee River just below the confluence of the French Broad and the Holston, and its relation to the watersheds of these two rivers is shown by figure 63, page 79. The greatest known flood on the Tennessee, that of March 1867, would be lowered about 8 feet by this regulation. The city continues to be plagued by floods on the small streams that flow through it. Three streams, First, Second, and Third Creeks, cause damage in Knoxville, and of these First Creek is the most important.

First Creek drains an area of 15.7 square miles above the city limits of which the upper 4.5 square miles apparently drain underground through sink holes. An additional 6.4 square miles drain into the creek as it flows through the city. The creek rises on the slopes of several low ridges north and east of Fountain City, a suburb of Knoxville. Entering Knoxville at its northernmost limit, near Whittle Springs, and flowing almost due south from Whittle Springs through a heavily populated area, the stream enters Fort Loudoun Lake just above the Gay Street Bridge.

The First Creek channel over most of its length in the city is obstructed by bridges, buildings, and fills. Debris has been thrown into it, and vegetation has been permitted to grow up on the banks. With the high runoff from 60 percent of urban drainage area, the creek overflows its banks annually or oftener, causing inconvenience and damage to a number of low-cost homes, commercial establishments, and industries. Near the lower end of the creek a channel improvement project completed in 1958 has eliminated the flood problem through a congested section of the city.

The greatest flood in the past 90 years on First Creek downstream to Fifth Avenue occurred on September 30, 1944. Raised by a 25-hour rain of 7 to 11 inches over the watershed, the creek overflowed 250 acres of its flood plain within the city, driving 150 families from their homes, flooding approximately 45 business places and several industries, blocking numerous streets, putting streetcars out of service, and generally upsetting the lives of hundreds of people. Flood history investigations and a search of newspaper files show that many other very large floods occurred, among them those of 1867, 1875, 1882, 1886, 1897, 1912, 1917, 1928, 1939, and 1948. The flood of June 29, 1928, was about the same size as that of 1944 above Fifth Avenue and exceeded the 1944 crest below this point.

None of these floods approached the maximum that might be expected from the First Creek watershed. Much larger floods have occurred on streams of similar physical characteristics in the region of Knoxville. Considering the discharges of these floods that are known to have occurred on streams in the region, it would be reasonable to expect a future flood on First Creek with a peak discharge four or more times that of September 1944. Such a flood would be at least 5 feet higher than the 1944 crest at Fifth Avenue and would cover substantially more area through the city above Fifth Avenue.

The 1944 flood caused tangible damage estimated at about $134,000, based on values at that time. On the basis of 1958 values, this loss would be raised to approximately $300,000. The February 1948 flood, which crested 1.5 to 3 feet lower than the flood in 1944, caused losses totaling $37,000, or about $60,000 at 1958 prices. A future flood 5 feet or more higher than that of 1944 would reach high-value property never before flooded and cause tremendous losses.

The channel improvements along First Creek, constructed by the city of Knoxville and the Knoxville Housing Authority in connection with the Riverfront-Willow Street Redevelopment project, extend from a point about 300 feet downstream from Vine Avenue up to Magnolia Avenue, a distance of 6,720 feet. For nearly half this distance the waterway is a concrete conduit and the remainder is a concrete-lined channel. Below the improvement the slope of the stream bed is steep and the natural channel provides a good waterway for flood flows. For more than a block upstream from Magnolia Avenue, the city has enlarged the channel by excavation but has not lined it.

The improved concrete channel was designed to carry a flow of 10,000 cubic feet per second, five times the 1944 peak discharge. This improvement greatly reduces the danger of overflows below Magnolia Avenue but does not change the flood situation in the upper reaches of the creek in the city.

Lake City, Tennessee

This small town of 1,914 people at the time of the 1960 census was formerly called Coal Creek after the Clinch River tributary that flows through it. The name was changed to Lake City on April 1, 1939, two years after the completion of Norris Dam 4 miles away. The livelihood of the residents depends principally on the coal mining industry. Lake City is shown on the map of the Clinch River watershed (fig. 68, page 88).

Coal Creek rises on the slopes of the Cumberland Plateau escarpment and flows generally eastward to enter Clinch River 5 miles below Norris Dam. It drains a fan-shaped watershed of 24.5 square miles above Lake City. The steep slopes are almost completely forested in second growth and scrub timber. The business district of Lake City is in the flood plain of the creek.

Two large floods, in March 1929 and in September 1944, are known to have submerged the business district of Lake City to depths up to 3 or 4 feet. Moderate floods occurred in February 1948 and January 1949. The 1929 flood resulted from the same great Cumberland Plateau storm that caused record-breaking stages on the Emory River watershed, a few miles to the west. Newspaper accounts stated that practically every residence and business place in the city was damaged by the 1929 overflow. Water was reported to have been 3 or 4 feet deep in many residences, and a store and cafe were practically demolished. The September 1944 flood, which was about 2 feet lower than in 1929, overflowed the business district to depths of 6 to 18 inches and other parts of the city to depths of 3 feet or more. Over 50 homes were flooded with water from 6 inches to 3 feet deep on the floors.

The peak discharge for the 1944 flood was estimated at 6,800 cubic feet per second or 280 cubic feet per second per square mile of drainage area. The peak discharge in 1929 was about 50 percent greater than that of 1944. Experience in the eastern part of the Tennessee Valley indicates that flows much larger than these might be expected from the watershed in the future.

The September 1944 flood caused damages totaling approximately $15,000 in Lake City, according to an investigation made after the flood. About three-fourths of this loss was to residential property and furnishings. At the time of the 1929 flood, newspaper accounts estimated the loss from that flood at $250,000.

In 1946-1947, the Corps of Engineers made a preliminary examination of the flood situation on Coal Creek. This examination showed a reasonable possibility of developing a feasible project, and a survey of flood protection was begun in September 1948. Early in 1951 the Corps recommended to the Congress a channel improvement plan extending downstream for 2.5 miles from the upper end of Lake City. The bottom width of the improved channel would be 100 feet. With this channel the 1929 flood stage at Main Street bridge would be reduced 8.2 feet or 1.2 feet below flood stage. Total Federal cost of the project would be $660,000 with local interests adding about $100,000. The project was approved, money was appropriated, work got underway in 1959, and the project is now (1960) complete.

Lewisburg, Tennessee

Lewisburg, county seat of Marshall County, Tennessee, a city of 6,338 people in 1960, is situated on the left bank of Big Rock Creek, a tributary which flows north into the Duck River (fig. 64, page 80). The low parts of the city are subject to periodic flooding from the creek. The greatest two known floods, of practically the same height, occurred in 1939 and 1955, with the latter causing damage estimated at $117,000. There were other high floods in 1902, 1912, 1926, and 1945.

The possibility of detention reservoirs, a new flood channel, and levees and walls for flood protection, were investigated. For all these plans the annual charges would exceed the damages prevented and for this reason it was not considered justifiable to provide flood protection for Lewisburg. The cheapest would be a simple levee about 3,000 feet long along the creek bank, but from the information available it appears that such a levee would not be possible for the entire distance, space is limited, and for much of the length the more expensive walls would be needed. Furthermore, either levee or walls would be undesirable because they would be some 6 feet above the level of the two bridges crossing the creek, making a barrier which would add considerable extra expense to get the streets across.

Pulaski, Tennessee

Pulaski, county seat of Giles County, Tennessee, a town of 6,616 people in 1960, is situated on the east bank of Richland Creek, a tributary which flows south into the Elk River (fig. 65, page 82). In its low-lying portions it is subject to damage from the floods of Richland Creek. In the flood of February 1948 the total damage was estimated to be $6,400. A flood in 1902 reached a point 6 feet higher than the 1948 flood, and if repeated now, such a flood would probably result in damage greater than $60,000.

About two-thirds of the damage might be averted by the construction of a levee which would cross the mouth of Pleasant Run, a small tributary in the eastern part of town, and would then run along the railroad. Highway U. S. 31 would have to be raised to cross this levee, and culverts and gates would be needed for internal drainage. Because the average annual benefit from such a project would be considerably less than the average annual cost it was not considered justified.

Roan Mountain, Tennessee

Roan Mountain is a village in Carter County, Tennessee, located on U. S. Route 19E between Elizabethton, Tennessee, and North Carolina points, and on the upper part of the Doe River which loops around two sides of the town. It is in the Holston River Basin (fig. 70, page 96). In high floods the lower parts of the town suffer damage not only from inundation but also from the high velocity of the water in this steep mountain stream. The greatest flood in history was that of 1901, in which the damage to this small town was high, although no estimate was made.

Studies indicated that the most obvious method of complete flood protection for Roan Mountain would be a flood bypass channel across the loop and behind the town. Detention reservoirs appeared to be impractical due to interference with highways, as well as cost. Some improvement in flood capacity could be made by widening and deepening the present channel. On a cost basis, however, neither the bypass channel nor the channel improvement could be justified.

Shelbyville, Tennessee

Shelbyville, county seat of Bedford County, Tennessee, is a city located on the Duck River about 221 miles above the mouth (fig. 64, page 80). In 1960 it had a population of 10,466. The greater part of the town is situated on high land above the reach of floods, but there is an area in the valley of Spring Creek, a small tributary in the middle of town, which is subject to inundation by backwater from floods in Duck River. This valley contains both residences and business property. The flood of February 1948 (fig. 122) caused damage in Shelbyville and vicinity estimated at $132,000, half of which was in the Spring Creek valley. There were even higher floods in 1929 and 1902, which if repeated now, would cause greater damage than that of 1948.

Shelbyville's only defense against floods at present is to attempt to minimize damage by organization of a disaster committee. This volunteer group, made up largely of local business men, acts with speed and efficiency in evacuating flooded families and their possessions ahead of flood crests. The local radio station cooperates with city officials in broadcasting flood warnings and information. In the January 1949 flood not a piece of furniture was damaged by the flood, and commercial losses were greatly reduced as a result of the activities of this organization.

In 1953 TVA studied the Shelbyville flood protection problem. A levee across the mouth of Spring Creek with either a creek diversion or a pumping plant was found to be the most practical plan to protect this area, but it was not economically feasible on the basis of existing development. Later the city proposed a redevelopment plan in the area. Early in 1959 the citizens of Shelbyville voted to authorize the city council to issue bonds up to $1,100,000 to finance the city's share in the Big Springs Urban Renewal and Flood Control project. This $8,000,000 project includes the clearing and redevelopment of 162 acres in the Big Spring area of the city, the construction of an earth fill levee to exclude backwater flooding by Duck River, and the installation of a pumping station to remove headwater flows within the protected area. The plan would provide protection against a flood 4 feet higher than that of 1902.

Sweetwater, Tennessee

Sweetwater, a town of 4,145 people in 1960, is located in the northwest corner of Monroe County, Tennessee. It is just west of the western edge of the Little Tennessee River watershed (fig. 69, page 92). Sweetwater Creek, a tributary of Watts Bar Reservoir, flows in a northerly direction around the south and east sides of the commercial and industrial district, and an area of some 12 blocks is low and subject to flooding from the creek. The damage in Sweetwater in the 1946 flood was about $18,600.

A detention reservoir above the town, while possible, would control only 30 percent of the drainage area and its estimated cost was much more than could be justified. A levee adequate to protect the flooded area would be about 3,500 feet long, would cross three roads, and perhaps cost more than could be justified for this small damage, although a survey would be required to make sure.

Sylva and Dillsboro, North Carolina

Sylva and Dillsboro are two nearby towns in Jackson County, North Carolina, with a combined population of 1704 in 1960. Sylva is the county seat. Dillsboro is located close to the mouth of Scott Creek, so that it suffers from floods of the Tuckasegee River as well as from the creek; Sylva is farther up Scott Creek and is flooded by the creek alone. The two towns are in the Little Tennessee River Basin and are shown on the map of that basin (fig. 69, page 92). The only known floods to cause appreciable damage in either town were those of 1840 and 1940.

Detention reservoirs did not appear to be feasible on either the Tuckasegee River or Scott Creek. In Dillsboro, furthermore, there appeared to be no location for a levee which would accomplish much protection. In Sylva, a levee could be built which would protect the damage area but, on a cost basis, it could not be justified.

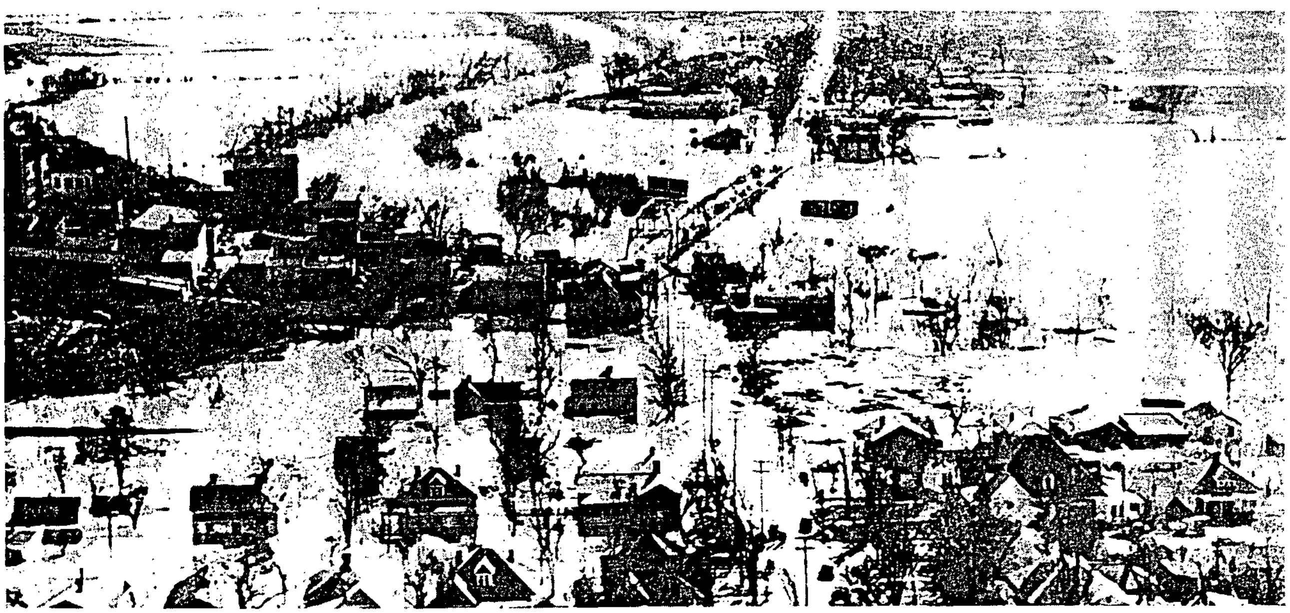


FIGURE 122.—*Shelbyville during 1948 flood.*

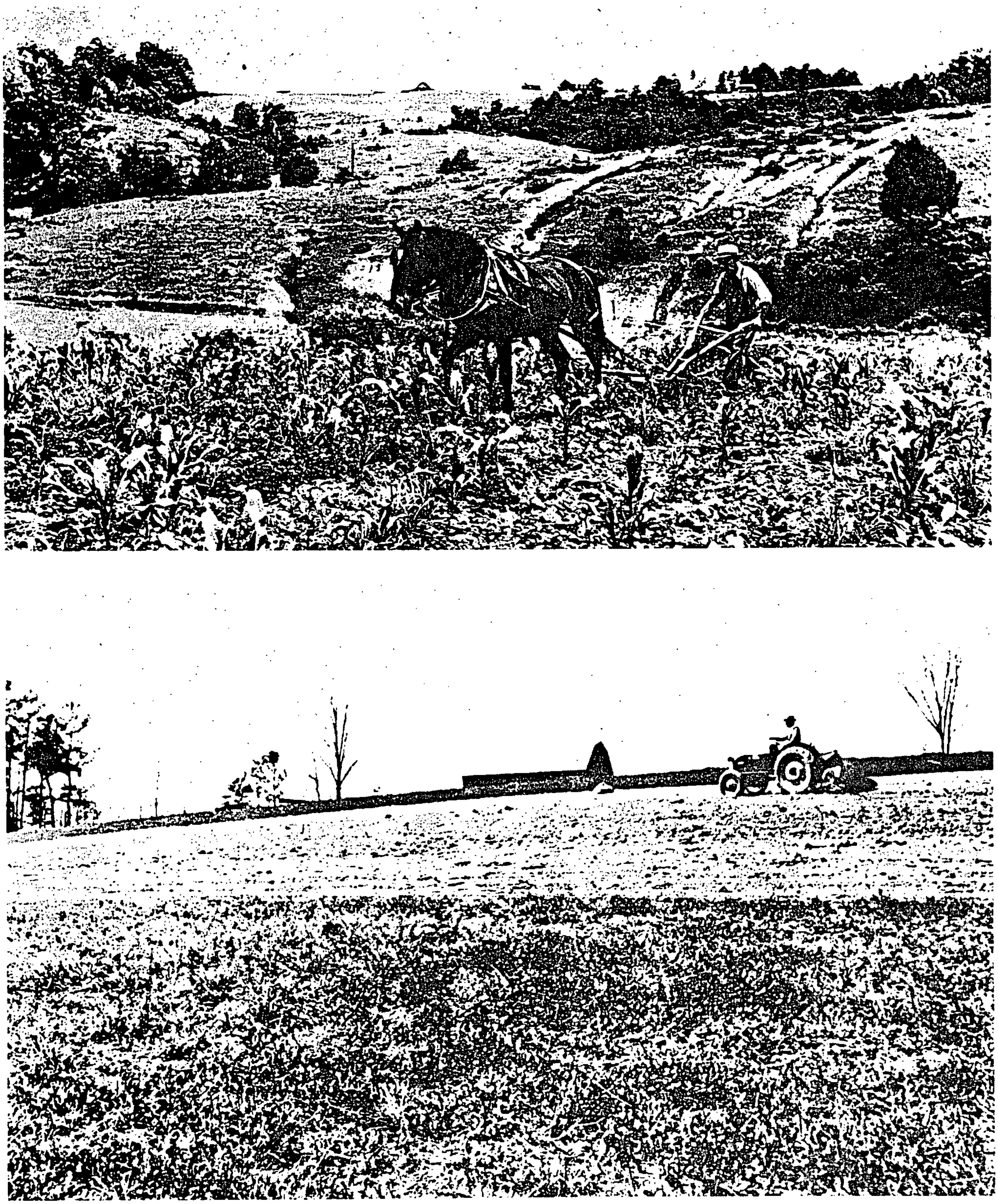


FIGURE 123.—*Changes in land use and management that have taken place in the Parker Branch pilot watershed research project—the horse, the plow, and the gullied land show the "before" condition, the tractor and the lush growth of alfalfa show the "after" condition.*

CHAPTER 11

EFFECT OF CHANGES IN LAND USE ON FLOODS

Along with the developments on the Tennessee River and its principal tributaries, TVA has been concerned with the improvement of the watershed itself. Congress directed that watershed improvement proceed concurrently with the major development of the river, and TVA therefore, has vigorously carried out watershed programs for improvement of agriculture and forestry. These programs—as they are related to the effect of changes in land use on floods—are described in this chapter. First, however, the chapter tells about various relationships between land use and floods, flood control, and erosion. Then, following the program descriptions, the chapter appraises land use effect on large areas using data from small watersheds, and discusses the present method for determining the influence on flood control of improved land use.

FLOOD CONTROL ASPECTS OF LAND USE

Improved land use may affect floods in two ways. The volume of runoff may be reduced by increasing the amount of water that can be stored in and on the soil, and the period of time during which the runoff enters the stream channel may be lengthened by the blocking action of vegetation or temporary storage in surface depressions. Both factors will serve to reduce the peak discharges of streams during floods.

The reduction in volume of runoff as a surface effect is related to increased interception of precipitation by vegetation, to storage of rainfall excess in stock ponds, and to the deterrent action of terraces and contour farming. Interception by a forest cover may amount to 0.5-0.75 inch during a storm. Interception by grasses may amount to ½ to ⅓ of this quantity. Interception by crops would be proportional to the ground cover provided, varying from zero at planting to complete cover at maturity.

The lengthening of the period during which runoff arrives at the channel is related to temporary storage, or detention, of overland flow. The chief factor is the reduction of velocities by vegetation, by forest litter, or by temporary surcharging of ponds, terraces, and farming contours.

Improved land use also produces indirectly a reduction in flood flow which is in addition to the effect of changing surface conditions. Some rainfall in each storm is absorbed by the soil. If the amount absorbed is greater than the soil can hold, it is lost slowly by gravity drainage. The moisture below the holding capacity of soil is depleted more slowly by evaporation and transpiration. Any changes in land use, therefore, which can increase the rate at which rain infiltrates the soil surface or can increase the storage capacity of the soil will serve to reduce flood peaks. These changes are produced both by reduction in the volume of runoff and by increasing the time of travel by passage through the soil rather than by direct overland flow.

Increased infiltration rates are associated with improved vegetal cover. Heavy vegetal cover retards surface flow and allows more time for water to enter the soil surface. The root systems of plants make the soil more pervious. Vegetal canopy shields the soil surface from rainfall and the destructive energy of rainfall impact is dissipated on the surfaces where it would otherwise cause breakdown of soil structure with consequent packing and crusting.

The storage capacity of the soil is related to both the porosity and depth of the soil cover of a watershed. Little can be done to increase soil depth, but proper land management practices will hold the rate of soil loss to a minimum and will preserve this reservoir. The porosity of the soil can be increased by increasing the organic content. Roots of plants penetrating to deep layers will break up tight soil horizons and the decay of roots will provide channels through the profile.

EXTREME FLOODS AND LAND USE

It would be desirable to relate the flood history of the Tennessee River and its major tributaries to a record of land use of the Tennessee Valley. Establishment of a clear relationship would serve as verification of the deductive conclusions and would allow estimates of quantitative effects in flood reduction by improved land use. Unfortunately, data do not exist to make this possible for all watersheds and for all flood conditions.

First settlements in the eastern Tennessee Valley took place prior to the Revolutionary War. The broad bottom lands were settled first, then the high coves were cleared, and finally the ridges also were cleared for farming. Unbroken stretches of virgin forest had to be cleared by these first settlers. There was no lumbering in the area and no transportation. Consequently, the forests were slashed and burned.

Destruction of the forest and farming of the steeply sloped soils resulted in rapid deterioration of land. Old fields were abandoned after short periods and new fields cleared by these early settlers. By the early 1860's, the process had reached a point where many of the mountain farms were being abandoned. It might appear that by the early part of the 19th Century the original forest cover over much of the Tennessee Valley would have been destroyed. Two factors, however, served to limit the amount of area cleared. There were no sawmills in the area other than for purely local markets before about 1870. Also, communications were very poor until after the War Between The States when the railroads first began to serve some of the remote areas. Because of little demand for lumber, and because each farmer cleared only enough land to grow the crops he needed for his own use, the early complete destruction of the cover was somewhat limited. Also, while worn out fields were abandoned in favor of newly cleared fertile areas, natural regeneration on these fields of even poor quality of cover would do much to retard rapid runoff and erosion.

Record floods occurred on the Tennessee River tributaries at times when there was still substantial forest cover. In April 1791 the highest flood known occurred on the Swannanoa River. The highest floods on the Clinch River were in 1826 and in 1840. Extensive flooding occurred on the Clinch, Holston, and French Broad Rivers in March 1847. In March 1867 occurred one of the greatest floods of record considering the large area of flooding. Record floods occurred on the lower French Broad, South Fork Holston, Holston, lower Little Tennessee, and the upper and middle reaches of the Tennessee River. Rainfall was estimated as 12.5 inches over a drainage area of 10,000 square miles.

The occurrence of these early floods is a seeming contradiction of the deductive expectation and this contradiction has been noted by hydrologists. While the deductive results have been verified by observation of runoff from small plots or experimental areas, they have failed on large areas. Jarvis recognized this controversy when he stated:[1]

> Such contrasts in surface runoff from small plots and experimental fields under a variety of cropping and land-use practices so far exceed the expectations of many practical-minded hydrologists as to invite challenge. They assert that the problems of water conservation and flood control cannot be resolved so readily as such experimental data would imply. They emphasize the generally accepted fact that the multiplication of respective yields from small plots by thousands or millions will not necessarily indicate accurately either the total flood volumes or the corresponding peaks as affected by surface conditions. Even the most ardent advocate of careful land-use and conservation practices as features of the flood-control program must admit the validity of such comments and reservations. There is some intermediate ground, however, on which the divergent views might reasonably be reconciled.

The benefits of good land use are most evident when considered beyond the limited field of record floods. Small floods are much more susceptible of modification by improved land use than are large floods of record-breaking magnitude.

Record floods on areas of appreciable size occur during the winter season as a result of the cumulative runoff of prolonged rains. During such storm occurrences the flood-reducing capabilities of even excellently managed lands are exhausted. Retardation of overland flow by a few hours is not highly significant in terms of storm runoff periods lasting for days. The soil profile becomes saturated and infiltration is limited to percolation through close textured subsoils while lateral subsurface flow increases and adds appreciably to streamflow. The amount of rainfall that can be withheld through interception and surface detention is a small percentage of the total during a storm which may deposit 10 or more inches of rainfall in a few days.

But potential small floods, during which the flood-reducing factors are not exhausted, can be reduced by improvement of these factors. The cumulative benefits of this reduction of small floods may well be of more economic importance in determination of flood-plain use and developments than the more severe but less frequent occurrences of record floods.

THE RELATION OF EROSION AND LAND USE TO FLOOD CONTROL

Every determination of effectiveness of improved land use as a flood control measure must contain an evaluation of indirect as well as direct effect. The degree of runoff reduction or flood peak reduction as a direct consequence of improved land use may be difficult to determine. But certain very real, if somewhat indirect, contributions to flood control by reduction of erosion are very evident.

One of the most important effects of a good vegetative cover is the protection of the soil from the damaging effects of rainfall impact. Sheet erosion caused by breakdown of the soil surface is virtually eliminated. Roots of the vegetation cover also serve to hold soil particles against the scouring action of overland flow. The net effect of a good protective cover is to hold the soil in place and prevent its transport to the stream channel.

1. "Hydrology" edited by O. E. Meinzer, Dover Publications, Inc., New York, 1949, p. 538.

This reduction in sediment has been observed under many conditions of cover improvement both in and outside the Tennessee Valley. Even establishment of poor quality of cover will serve to cause rapid reduction of sediment load in the streams. While reduction of peak flow has not been accomplished, vegetal cover has allowed the watershed to pass high rates of flow without physical damage of irreplaceable lost soil.

Retarding of erosion has an important effect on downstream flooding. High rates of erosion and sediment transport cause deterioration of stream channels and lost capacity of detention reservoirs by deposition of the sediment. Maintenance of channel and reservoir capacity is an indirect but important aspect of flood control in relation to land-use practices.

IMPROVED LAND USE EFFECT ON SMALL FLOODS

Information on historic floods is usually limited to streams draining large areas or streams that flow through developed regions. The spectacular nature, long duration, and areal extent of an extreme flood all serve to fix the event. But many smaller floods have occurred for which no records exist. These lesser floods may have occurred before the beginning of systematic record keeping or may have occurred on smaller streams where they have only local interest.

The total number of occurrences of small floods is much greater than occurrences of large destructive floods and is of more concern to those affected. It is probable that a person living along a stream will experience many small floods during his lifetime, but may never experience a large devastating one. Although the damage caused by each of these lesser floods is small, the total accumulative effect is large because of the extensive total area drained by the lesser streams and because of the more numerous occurrences of such floods.

Where streams flow through agricultural lands, the more numerous occurrences of small floods practically dictate the use which can be made of the flood plains along the streams. Where streams flow through urban areas, economic pressures have caused encroachment on the flood plain. In either case reduction or elimination of these small floods has economic benefits. In agricultural areas reduction of frequent small floods permits better use to be made of lands subject to such flooding. In urban areas damage by floods can be reduced in frequency or, where flood zoning is resulting in redevelopment, permissible uses of the flood plain may be modified.

EFFORTS OF TVA IN LAND USE IMPROVEMENT

TVA is concerned with improvement of land use in the watersheds of the Tennessee Valley. The TVA Act passed by Congress in 1933 provides for agricultural development of the region and for the "proper use, conservation, and development of the natural resources of the Tennessee River Basin." Expressly stated in the Act are "the proper use of marginal lands" and "the proper method of reforestation of all lands . . . suitable for reforestation." The activities of TVA under these directives from Congress have been concentrated in two programs, agricultural improvement and forestry improvement.

Agricultural improvement program

The improvement of agricultural lands is an important aspect of the total resource development of the Tennessee Valley. The protection of the watershed from erosion due to poor land management and the preservation of the soil and the improvement of its productivity are both necessary to the economic well-being of the individual farmer and to the economic level of the region.

TVA has developed an integrated program of research and application in its program of watershed protection and development. Research in chemistry and chemical engineering at the TVA Fertilizer-Munitions Development Center at Muscle Shoals, Alabama, has resulted in constant improvement in the quality of fertilizers and constant progress in lowering the costs of fertilizers to farmers. These fertilizers are tested on widely divergent soil types through cooperative arrangements with agricultural experiment stations. Land-grant colleges, private and cooperative fertilizer distributors, as well as other agricultural agencies cooperate in programs of test demonstration by which the individual farmers learn to use modern commercial fertilizers.

The hydrologic evaluation of the benefits derived from activities of TVA in agricultural improvement is based on investigation at specific tributary watershed projects described in the following paragraphs.

Chestuee Creek—This watershed is a typical agricultural watershed in east Tennessee. Measurements of runoff and sediment were made over a 10-year period in its 85,000-acre area. Concurrently with hydrologic measurements, work with the farmers in the area was carried on by agricultural extension service personnel with the objective of improving watershed cover and land management. Land use surveys were made at the beginning and the end of the period, with the watershed cover classified as cultivated, pasture, or forest. Each of these groups was further subdivided into classes, depending on the quality of cover.

Significant changes in land use took place in the Chestuee watershed between 1944 and 1954 when the surveys were made. Cropland was reduced from 34,543 acres to 20,734 acres. Pasture lands increased from 16,234 acres to 22,629 acres and forest lands increased from 32,048 acres to 32,994 acres. The reduction in croplands and increase in forest and pasture lands have significantly improved the protective cover of the watershed. Of equal importance, however, in hydrologic evaluation was the great improvement within the respective classifications. There were striking increases in the better classes of pasture and forest and decreases in the poorer classes of cropland, pasture, and forest. For example, there was a reduction of 56 percent in the poorer quality, less-well-managed cropland, while the first three classes of pasture increased 162, 39, and 74 percent, respectively.

Analyses of suspended sediment measurements which were obtained from 1944-1954 show significant reductions in sediment loads. For the 10-year period, the annual suspended sediment load decreased 48 percent for the entire watershed. Reductions of 45 and 32 percent occurred for two subportions of the watershed. This reduction in suspended sediment is representative of hydrologic changes which can be produced by improved land cover. Complete analyses of data will in all probability similarly indicate reductions in peak discharge and possibly volume of runoff.

Western North Carolina Cooperative Watersheds—These are research areas of 3.5 to 5.6 acres near Waynesville where TVA and North Carolina State College of Agriculture and Engineering are cooperating in investigations of basic soil-water relationships. An important phase of this project is the study of changes in peak discharge and volume of runoff under different agricultural covers. Measurements of soil moisture and evapo-transpiration are also being made and physical characteristics of the soil under different covers are being determined. The following purposes are included in the work plan for the project:

> The work under this project is restricted to the agricultural lands of the mountainous region of Western North Carolina. Soils of this area are subject to heavy erosion and high runoff after removal of the native forest cover. Consequently, stream channels are silted and bottom land overflows become more frequent. Permanent loss of productive topsoil occurs, and the water retained in the soils available for crop production is reduced. This research in Western North Carolina will stress the effects of pasture management and principal vegetative covers on the hydrologic characteristics of these soils. Such data are a prerequisite to the development of a program of land use for this area that will provide maximum utilization of the rainfall.

The Western North Carolina project was initiated in 1948. It was originally designed as paired watersheds in which one watershed would serve as a control while the effect of cover change on the other was measured. Data accumulated to 1952 revealed that the presumably paired watersheds were not hydrologically identical or even similar. In August 1952 the cooperating agencies adopted a Latin Square statistical design for operation of the project. By this plan, four agriculturally important vegetative covers are rotated on four of the watersheds. These covers are (1) corn; (2) wheat, with lespedeza for summer cover; (3) improved clover-grass pasture; and (4) improved pasture sod which is overgrazed and heavily trampled. The covers are scheduled among the four watersheds so that at some time in the project life each cover will follow each of the other three covers. At the end of the project it will be possible to analyze (1) the effect of carry-over from one crop to another, (2) the effect of varying watershed characteristics, and (3) the effect of each crop on the hydrology.

While this project has not advanced to the point where final conclusions can be drawn, preliminary results are striking. Peak discharges from the watersheds which have been observed correlate excellently with the cover conditions. Table 33 shows the average of the highest five peak rates of discharge which have been observed from all covers on the watersheds which are now in the Latin Square design.

The highest peaks which have been observed came from land prepared for corn but with little cover. These peaks averaged about six times the peaks observed from improved pasture. The highest peaks from the relatively poor covers of overgrazed pasture and broom sedge were not much greater than from the improved pasture. However, the peaks from corn, wheat, and transition, where soil disturbance was involved, were considerably larger. Independent statistical testing of these peaks in relation to the rainfall has shown that these major differences are significant.

Studies made to date of total volume of runoff from the watersheds have not shown any changes due to the different covers. Changes in infiltration rates, however, are apparently developing, and the continuing accumulation of data should serve to verify this preliminary observation.

In addition to the results of studies of peak discharge rates and volumes of runoff some preliminary results have been obtained from studies of soils characteristics. Under the condition of over-grazing, significant compaction of the top layers of soil resulted from excessive trampling. There was an

TABLE 33.—*Western North Carolina cooperative project—average of highest five peak rates of discharge, by covers.*

Cover	Number of peaks	Average peak, cubic feet per second per acre
Corn	10	1.82
Wheat	10	1.64
Broom sedge	5	0.58
Transition [1]	5	1.37
Overgrazed pasture	5	0.42
Improved pasture	20	0.30

1. Change from broom sedge to pasture with spotty cover.

associated decrease in the capacity of the soil to retain and to conduct water. Soil moisture studies are also producing significant information on the amount of moisture reservoir available under the different land uses. Continuing studies of all information being obtained from this project will serve to produce an integrated result of the entire soils-water relationship.

Parker Branch—This pilot watershed research project in Western North Carolina is also being conducted in cooperation with North Carolina State College of Agriculture and Engineering. The watershed has an area of 1060 acres. The objectives of this project are to determine the effects on the hydrology of the area of an intensive farm development program designed to produce maximum economic well-being of the people using the land.

Hydrologic measurements on this project include precipitation, streamflow, and stream suspended sediment and deposited sediment. Project activities are divided into three chronological phases of calibration, action, and evaluation. During the calibration phase the hydrologic observations provided a base against which future changes could be measured. The observations were continued through the action phase, during which extensive renovation of the watershed was accomplished in a short time. Heavy equipment was used to move earth to fill gullies. Extensive improvements were made in the quality and type of watershed cover. All physical renovation and cover improvements were the result of farm management plans developed for maximum economic well-being. They were not the result of a program of maximum conservation, but best economic use of land did work out to provide a large measure of hydrologically effective watershed cover.

Detailed soils-land use inventories of the Parker Branch watershed have been made at intervals. Comparison of land use during a survey made in August 1957, near the end of the action phase, shows some striking differences from land use during a survey made in March 1953, near the beginning of the project (fig. 123, page 202). Between these two dates there was little change in total area in cropland, which amounts to about 46 percent of the watershed. There was an increase, amounting to 18 percent of the watershed area, in the cropland in the two best of four cropland classes. The total area of forest decreased by an amount equal to six percent of the watershed. Most of this was a reduction in the poorest forest class. The reduction in forest land is about balanced by an increase in pasture. In 1957 pasture covered 29 percent of the watershed, an increase amounting to five percent of the watershed since 1953. There was a significant improvement in the quality of pasture. The amount of land in the two best of four pasture classes increased by an amount equal to 17 percent of the total watershed area.

Final evaluation of the hydrology of Parker Branch under a program of maximum economic return must await the conclusion of the evaluation phase of the project in 1962. However, results at the end of the action phase show already that maximum economic return is compatible with watershed protection. Analyses to that point show that peak discharges during the winter season have been reduced. Suspended sediment loads have been reduced both in summer and winter season. The character of deposited sediment has changed from a fine silty material to a coarse sandy deposit. Continuing studies will be necessary to decide whether there has been any change in storm runoff or infiltration rates.

Forestry improvement program

The effort of TVA to evaluate by hydrologic measurements the effect of improvement in forest cover has been concentrated in two research watersheds. These watersheds—Pine Tree Branch and White Hollow—are not similar in their project objectives but both have provided conclusive results.

Pine Tree Branch—This is an 88-acre watershed near the western edge of the Tennessee Valley. The watershed is located in and is typical of an area in which serious problems of water control, erosion, and land use exist. At the beginning of the investigation the watershed was in a severely eroded condition. There were numerous narrow gullies with almost vertical sides penetrating the subsoil and substratum to a maximum depth of 10 feet. Severe sheet erosion had carried away most of the original surface layers.

The Pine Tree Branch watershed was maintained in its initial condition for calibration purposes from the beginning of the project in 1941 until 1945. During this initial phase the watershed had a poor cover consisting of 23 percent forest, 9 percent pasture, 16 percent cultivated, 50 percent abandoned and idle land, and 2 percent miscellaneous. A program of watershed improvement was then carried out, embracing tree planting, contour furrowing, diversion ditching, and check dams in the channels and gullies. In 1950 the watershed was classified as 100 percent forest, with 85 percent of the area having cover better than 95 percent.

Hydrologic measurements were maintained through the 10-year period. The following effects of cover improvement and erosion control were noted:

A. *Water yield*
   1. Surface runoff volumes decreased and ground-water discharge increased.
   2. There is some indication of a slow, progressive decrease in water yield.

B. *Evapo-transpiration plus other losses*
   1. Evapo-transpiration plus other losses have apparently increased slightly.
   2. Greater interception by improved vegetal cover is a factor in the increase in evapo-transpiration plus other losses.

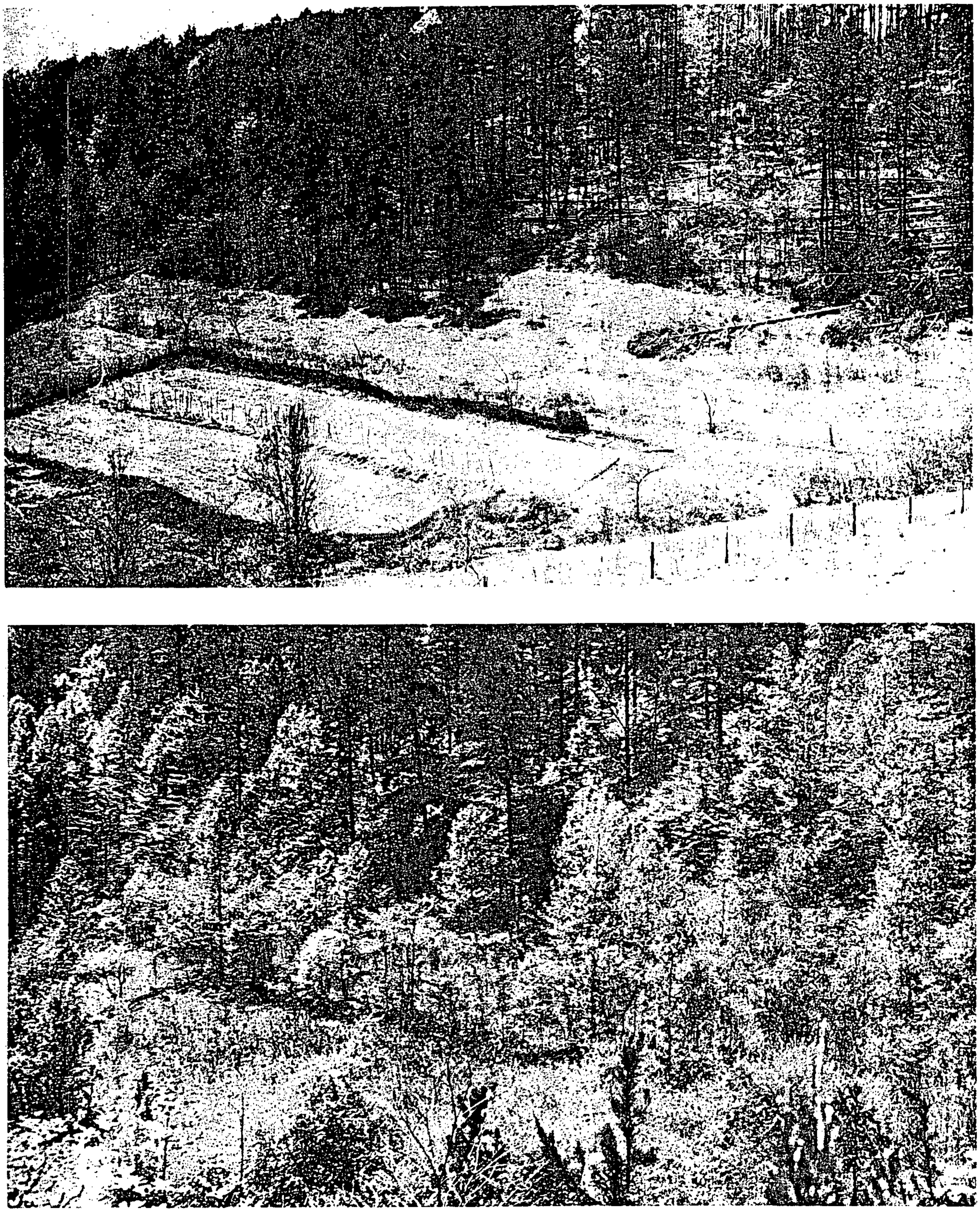


FIGURE 124.—*White Hollow watershed—top view taken in 1935 and bottom view in 1946 from approximately same location.*

C. *Streamflow*
1. Peak discharges in both summer and winter floods have been markedly reduced. Peak rates of comparable moderate to large storms in the period before treatment averaged about 3 to 4 times those in the period after treatment, with variations depending largely upon antecedent soil moisture condition.
2. Surface runoff volumes from individual storms have been reduced. Those volumes before treatment, were from 1½ to 4 times those after treatment, with variations depending upon rainfall amounts and antecedent soil moisture conditions.
3. Reductions of surface runoff volumes in both winter and summer storms are substantially the same. This equality of reduction is influenced by year-round effect of a coniferous cover.
4. The time-distribution pattern of surface runoff shows a noticeable reduction in overland surface velocities under present conditions.
5. Higher storage characteristics resulting from check dams and vegetation in stream channels prolong the period in which surface runoff drains out of the channel system after overland flow has ceased.
6. There has been no change in the recession of ground-water flow.

D. *Sediment*
1. The rate of sediment removal has progressively decreased to a fraction of the amount at the beginning of the investigation. Compared with the average rate of stream total sediment load during the pre-treatment years, 1942-1945, the sediment rate in 1946-1947 was reduced by 46 percent, in 1948-1949 by 77 percent, and in 1950 by 90 percent of the original rate.

E. *Ground-water levels*
1. No detectable change was observed in the annual cycle of ground-water levels near the main stream that could be ascribed to the watershed treatment.

White Hollow—This is an area of 1715 acres tributary to Norris Reservoir and was part of the land acquisition program for that project. In 1934 the families who resided there were moved out and the area was taken out of cultivation. At that time the soils of the watershed were generally severely eroded and gullies were numerous and active as a result of clearing and cultivation of almost the entire area during the 150 years since the first settlements.

The acquisition by TVA and the subsequent placing of the area under forest protection and management presented an opportunity to study the effect on the hydrologic factors of runoff and erosion from a watershed taken out of cultivation. Under forest management and protection during the 21 years covered by the study, plantings and natural revegetation have resulted in a continuously improved vegetal protective cover over the watershed. In 1936 a survey of cover conditions on the watershed showed 66 percent forest, 4 percent cultivated, 4 percent grass, and 26 percent abandoned land. A resurvey in 1946 showed all the area in forest except a few areas of less than an acre at the old home sites. Figure 124 shows two views taken in the White Hollow watershed from approximately the same location but about 10 years apart—in 1935 and 1946.

Measurements of precipitation, streamflow, and sediment during the 21-year study period show the following:

A. *Water yield*
1. No appreciable change has occurred in the water yield from the watershed.
2. The improvement in forest cover which has occurred has resulted in greater watershed protection without measurable decrease in water yield.
3. There has been no shift in the seasonal runoff pattern as a result of land-use changes. The principal controls upon runoff continue to be precipitation amount and season.

B. *Evapo-transpiration plus other losses*
1. No measurable change has taken place in the total quantity of evapo-transpiration plus other losses.
2. The effect of greater shading, reduced wind velocity and greater humidity has evidently resulted in reductions in evaporation approximately equal to the consumptive use by plants.

C. *Streamflow*
1. Peak discharges during the summer season have been markedly reduced. The discharge rates have been reduced for comparable storms to an amount only 5 to 27 percent of those initially observed.
2. Winter peak discharge reductions have been less than in summer, and such peaks for comparable storms are 72 to nearly 100 percent of those initially observed.
3. The greater part of the summer peak discharge reduction occurred in the first two or three years of investigation, smaller reductions continuing after that time. This fact is significant in indicating the rapidity with which peak flow reduction can be effected by vegetal cover.
4. The time distribution of surface runoff has been materially changed. Surface runoff discharge has been prolonged to produce a more sustained flow.
5. Recession of runoff draining out of the watershed channel system after overland flow has ceased is delayed as the result of higher storage characteristics developed within the storage channel itself.
6. No change due to increasing vegetal cover has been observed in the rate of recession of ground water.

7. There has been no change in the amount of either surface and sub-surface runoff or ground-water runoff.

D. *Sediment load*

1. In 1935, the total sediment load from White Hollow watershed, including both suspended and bed load was about 15 times that in 1952, no significant additional reductions occurring to 1955. The greatest reduction in sediment rate occurred in the early years. For the water-year October 1935-September 1936 the sediment load for the average storm was 7.3 tons. By the water-year 1951-1952, the load was only 0.5 ton, a reduction of 93 percent.

APPRAISAL OF LAND USE EFFECT ON LARGE AREAS FROM SMALL WATERSHED DATA

The broad scope of TVA activities in watershed improvement demonstrates clearly the reduction in peak discharges and sediment loads which can be obtained on relatively small areas. The reduction in floods which would result from a reduction in volumes of runoff is less clearly demonstrated, but is still real on some relatively small areas.

However, the effect of land cover improvement on floods from large watersheds is still not susceptible to conclusive determination. There is at present no satisfactory method by which the hydrologic characteristics of peak discharge, volume of runoff, and infiltration, as determined from small areas, can be extrapolated to produce a consistently verifiable effect from large areas. Some promising techniques have been developed, but much research needs to be done in this field. For example, the development of universal quantitative soil-moisture relations is necessary, as well as the development of methods to apply these relations to ungaged areas and produce realistic integral results for fantastically variable combinations of soil, cover, management, and meteorological factors.

There are two current fields—discussed in the following paragraphs—where TVA is attempting to establish improved means of applying the results of small watershed investigations to large watersheds.

The Cooperative Research Project in Western North Carolina is providing basic information on the variability of soil-moisture relations for some soils-cover complexes. Part of the purpose of this project is to provide data which "are a prerequisite to the development of a program of land use for this area that will provide maximum utilization of rainfall." The maximum utilization of rainfall for agriculture will, of course, reduce to some minimum the amount of rainfall which becomes flood runoff.

Beech River watershed in western Tennessee, will provide basic information which can be used to help develop methods for integrating the effects of land use on large areas. Beech River watershed includes an area of 302 square miles of mixed agricultural, forest, and idle lands located on the western edge of the Tennessee Valley in west central Tennessee. The stream pattern consists of one main channel, Beech River, which flows easterly into Kentucky Reservoir, and numerous tributaries ranging up to 23 square miles in drainage area. In this area TVA is cooperating with existing Federal, state, and local agencies in development of the relationships among the use of agricultural resources in the watershed, the hydrologic conditions within and beyond the watershed, and the welfare of the people who use the watershed resources.

Measurements of precipitation, streamflow, and suspended sediment loads on Beech River and on several subdivisions of the watershed have been made since January 1953. These, and future measurements, will provide detailed information about the hydrologic characteristics and changes in the watershed and its subdivisions that result from land-use improvements. Land-use surveys were made in portions of the watershed in 1950, 1954, and 1957.

The scope of the Beech River watershed project offers a unique opportunity for comparative hydrologic analyses. Hydrologic information from the instrumented subdivisions of the watershed can be correlated with other information on soils and land use. Some study can also be made of the way in which the subdivisions contribute to the hydrologic characteristics of the entire watershed.

PRESENT METHOD FOR DETERMINING INFLUENCE OF IMPROVED LAND USE ON FLOOD CONTROL

Engineering problems arise in planning flood control or watershed development projects where it is necessary that some estimate be made of the effect of land use changes. Present limited methodology and knowledge must then be supplemented by sound engineering judgment in arriving at firm estimates.

TVA has developed a method of estimating effect of land use changes based upon classifying lands by infiltration capacities. This method was developed during the preparation of the plan for flood control for the upper French Broad River. The method was subsequently used in the initial studies for the Chestuee Creek watershed project and later it was used to compute the influence of improved land use on the hydrology of Turkey Creek in the Beech River watershed.

Under the method developed for the upper French Broad planning, all lands are designated as forest, pasture, or cultivated, and three or four subclasses for each designation are defined on the basis of infiltration capacity. The original classifications were chosen to make use of experimental data from the Appalachian Forest Experiment Station at Bent Creek, North Carolina, and from the Virginia Agricultural Experiment Station near Blacksburg, Vir-

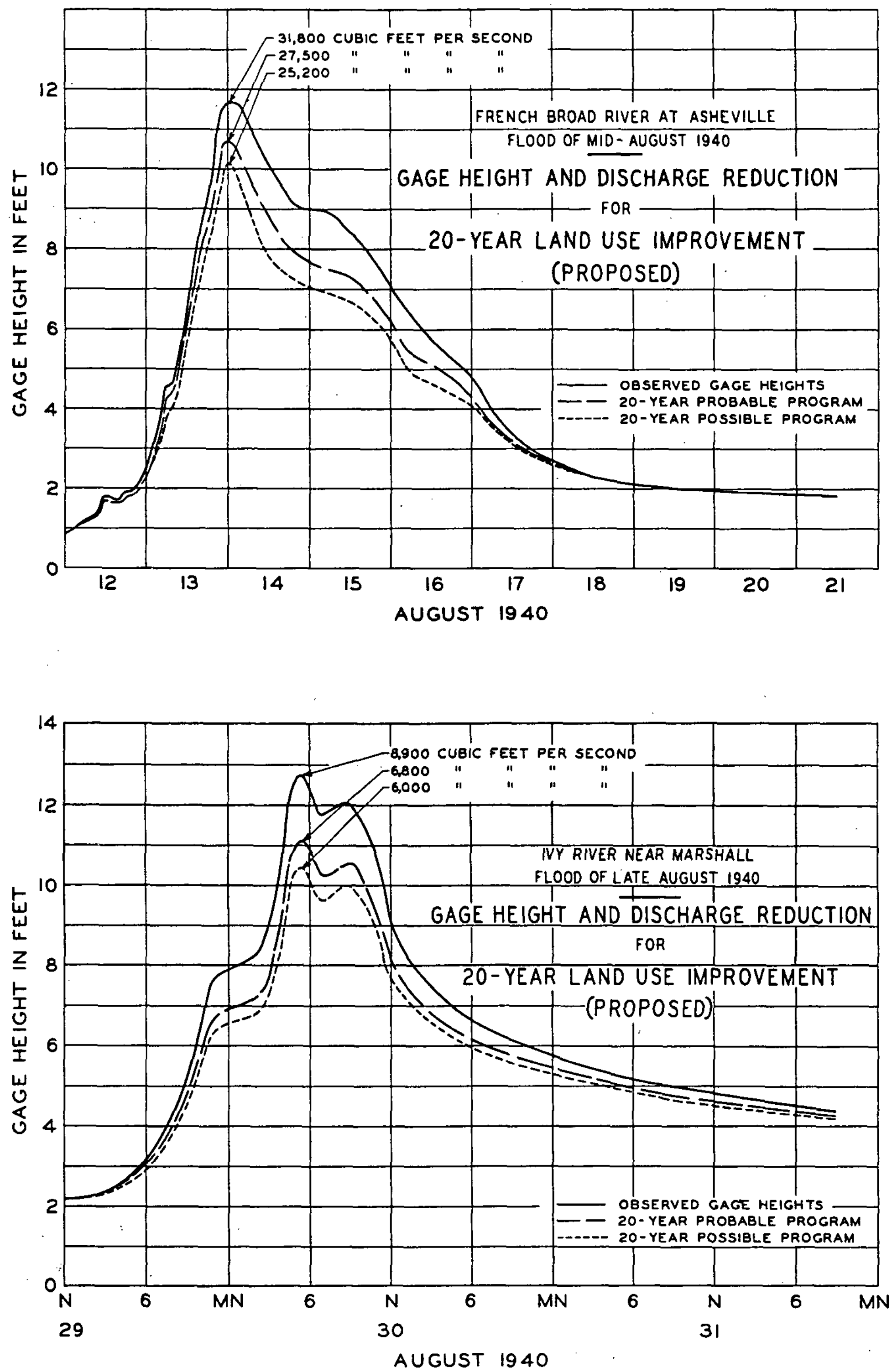


FIGURE 125.—*Effect on floods of projected land use programs.*

ginia. Initially, experimental infiltration rates were determined by analysis of stream hydrographs and rainfall patterns for data from each cover classification for which experimental data were available.

The experimentally derived infiltration rates were used to obtain theoretical infiltration rates for selected unit areas in the French Broad watershed. The experimental infiltration rates for the eleven cover classes were weighted in the proportion that each land class existed as determined by land use surveys of the selected unit areas. Actual infiltration rates for the unit areas were also computed by analysis of stream hydrographs and rainfall patterns, in the same way that the experimental data had been analyzed. Comparison of the theoretical infiltration rates and actual infiltration rates from the unit areas was then possible. In order to make the two rates agree, it was necessary to reduce the theoretical rates by 20 percent. In this way the experimental data for each of the eleven infiltration classes were adjusted to the observed data from the upper French Broad watershed.

The North Carolina State College Extension Service and the TVA Forestry Relations Division developed programs for land use improvement for periods of 5, 10, 15, and 20 years in the future for the open lands and forest lands, respectively. These programs were based in part upon actual improvements over a 5-year period on Unit Test Demonstration Farms in the watersheds. Two programs were formulated. One was called a "possible program" and was based on maximum improvement. The second was called a "probable program" and assumed a less intense development and one that might be expected to be accomplished practicably.

The data from the two development programs were used to determine future average infiltration rates. The computations were based on two storms which occurred in August 1940. The observed average infiltration rates were increased by the same amounts as it was estimated that the 20-year land improvement would increase the theoretical 1940 infiltration rates which were based on the land use classification. These projected infiltration rates were applied to the storm rainfall pattern and reduced amounts of rainfall excess were computed. The ordinates of the August 1940 storms were then reduced by the ratio of the 20-year program excess to the actual 1940 excess.

The computation of reduced hydrographs for the August 1940 storms was accomplished on each of the subwatersheds of the upper French Broad. The reduced hydrograph for Asheville, North Carolina, was computed by routing the reduced subwatershed hydrographs through the main-river reaches, giving proper consideration to reductions in local inflows and changes in ground-water flows.

Results of the determination of the effect of the projected land use programs are shown in figure 125. This shows the reductions which were computed for the French Broad River at Asheville for the mid-August 1940 flood and for the Ivy River near Marshall for the late August flood for both the possible and probable programs of land use improvement.

CHAPTER 12

DAMAGE FROM FLOODS

The growth of flood damage as flood plain development intensifies is briefly traced in the first few paragraphs of this chapter. It then classifies flood damages and outlines the bases of TVA's damage estimates, discusses potential flood damages in Chattanooga in relation to periodic appraisals for that city, explains how annual preventable flood damages at critical locations are determined, and summarizes these damages. A brief discussion of future flood damage ends the chapter.

FLOODS AND FLOOD PLAIN DEVELOPMENT

Although floods have always occurred, serious flood damage to man and property has resulted only because of man's occupation and use of the flood plain. Previous to such occupation, flooding was generally beneficial to future use because of the deposition of silt in the valleys. The relatively level plains thus formed provided easily cleared and fertile land suitable for cultivation, as well as the most attractive sites for urban centers. Moreover, the stream was a source of water supply and about the only major means of transportation. The advantages of a community location on a stream usually outweighed the disadvantages of occasional inconvenience from floods.

As the communities increased in size and population, the value of property also increased until the time arrived when great damage was caused by floods of a magnitude which formerly produced only minor inconvenience. In many communities which were thus unfortunately located with respect to flood damage, the investments have become so large that relocation is not now feasible. Nevertheless, the inhabitants must recognize the fact that most stream channels and overflow areas were created by Nature to be used for the passage of water or for temporary storage during floods. They should become reconciled to the fact that if they continue to occupy those overflow areas, flood damage and loss of life will be inevitable unless protective works are provided.

Settlement along the Tennessee River and its tributaries followed a pattern similar to that in other parts of the country. As a result, the flood hazards in the Tennessee Valley ranged from flooding of farm lands, with loss of crops or livestock, to the extensive and serious flooding of urban centers with great damage to residential and industrial property and possible loss of life. Villages which grew into large cities, a portion of which are on low-lying flood plains, include Chattanooga, Asheville, and Kingsport.

As the population of the Tennessee Valley increases, the pressure for further use of the flood plain also increases. Moreover, because of partial flood regulation afforded by TVA reservoirs, there will be increasing use of land which now is flooded less frequently than before. Industries will be attracted to the favorable locations on the flood plain for the additional reasons of low-cost water transportation and ample water and power supply. Land which formerly was flooded too often for profitable farming now may be cultivated. The effect of this increased land use will be, of course, to increase the potential flood hazards along the rivers.

FLOOD DAMAGE CLASSIFICATIONS AND ESTIMATES

Flood damages may be classified according to geographical location; the time of the year when flooding usually occurs; the type of affected property; whether the damage is of regional or local nature; whether the property is publicly or privately owned; and whether the losses are direct property damage or indirect losses. A check list of losses provides a convenient means for ensuring a complete tabulation and analysis of all losses. Such a list has been prepared by the National Resources Committee.[1] A more detailed discussion and classification of flood control benefits as related to various kinds of damages resulting from floods is given in chapter 13.

Only estimates of primary, tangible, preventable damages such as direct damage to all types of physical property, land, and crops; estimates of indirect losses such as those of wages, industrial output, retail sales profit, and public utilities income; and relief expenditures are used as the basis for estimating benefits due to the TVA reservoir system. As discussed in chapter 13, secondary tangible benefits due to increased property values along the lower Ohio and Mississippi Rivers due to prevention of overflow of levees are included in the economic comparison. Intangibles, such as the prevention of the loss of life, are not included.

In the following discussions, estimates of annual flood damage are made for the cities and towns which receive some degree of protection from the present TVA multiple-purpose reservoir system, and

1. National Resources Committee. "Report of the Subcommittee on Flood Damage Data," March 15, 1939 (mimeographed).

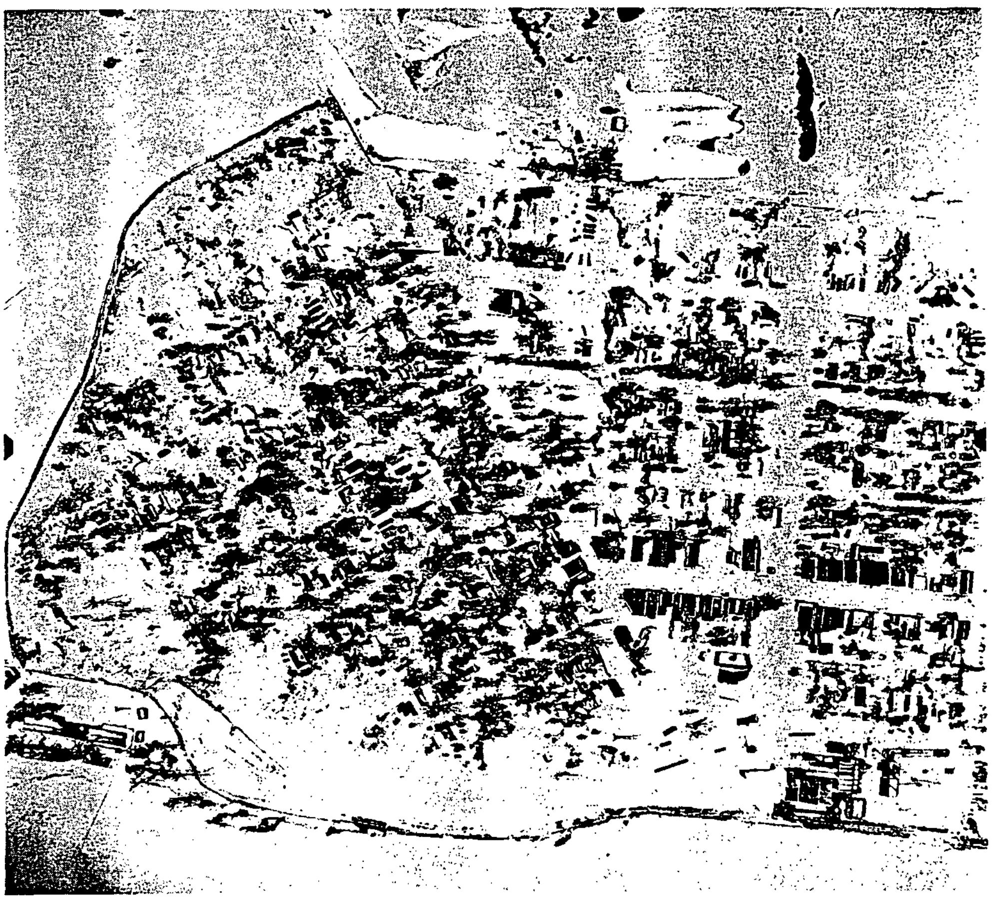


FIGURE 126.—*Top view: Looking across Chattanooga from Lookout Mountain during 1917 flood (Cline Photo Collection, Chattanooga). Bottom view: Mound City, Illinois—on Ohio River downstream from mouth of Tennessee—during 1937 flood.*

also for the agricultural land lying downstream from the tributary reservoirs and between main-river normal pool levels and the maximum summer flood level. The estimate for Chattanooga was made after thorough studies of flood heights and field surveys of affected property in 1938, 1948, and 1953. Indirect losses were included in this estimate. For smaller towns on or near the Tennessee River, including Clinton on the Clinch River, flood damage estimates are based largely on the number of affected buildings as counted from maps, and a unit damage for houses determined from the Chattanooga surveys. Estimates of agricultural damage are based on the area protected from floods occurring from May to November, using unit damages per acre based on known values at other locations. A small amount is included in the total to cover damages at other towns and to industries located outside corporate limits. No intangible damages are included in any of the estimates.

POTENTIAL FLOOD DAMAGE IN CHATTANOOGA

1938 appraisal

In connection with the preparation of the report, "The Chattanooga Flood Control Problem," (House Document No. 91, 76th Congress, 1st Session, 1939) a field survey of potential flood damages was made in 1938. The survey included areas on both banks of the Tennessee River and extended into the contiguous community of Rossville, Georgia, but excluded other large areas within the corporate limits of Chattanooga because they would not be affected by the proposed levee system, particularly that portion of the city east of Missionary Ridge. This report states that the purpose of the appraisal ". . . was to determine (1) the flood damage to the present (1938) city of Chattanooga if the highest recorded river stage were repeated, and (2) the damage which would now (1938) be caused by lesser floods, so that a calculation could be made of the average annual flood damage which would be caused by a repetition of the floods from 1867 to 1938 during a period of the same length and with improvements as they are at present."

The methods used in making the Chattanooga appraisal, as contained in the above-mentioned report, are included herein as Appendix C. A field count of all properties affected by floods reaching heights of 58 feet (1867 flood), 53 feet, and 48 feet was made, and the properties were classified as to residential, commercial, or industrial use. Damage to residential property was estimated from the type of construction, number of stories and rooms, and depth of flooding. Unit damage rates were based in part on results of a house-to-house survey of the actual damage caused by the 1937 flood at Paducah, Kentucky. Based on experience in other cities, it was assumed that commercial establishments flooded would suffer practically a complete loss of stock and fixtures. Separate appraisals were made for each industry with the aid of the owner or manager if possible. Indirect damages were computed for loss of wages, loss of industrial output, as represented by the value of products less manufacturing costs, loss of profit on retail sales, loss of receipts of utilities, and relief expenditures.

Total damages at the three stages of 58, 53, and 48 feet were appraised at $37,656,000; $21,612,000; and $9,029,000, respectively. Since, at the time of the survey, damage began at about a stage of 33 feet, four points were thus available for constructing the curve designated in figure 127, "City as of 1938."

1948 appraisal

The great expansion of industrial activity in Chattanooga between 1942 and 1945 (during World War II) and the further development of all kinds after the war indicated the need for a reappraisal of the flood damage. Accordingly, in 1948 a survey was made of all the new development that had taken place between 1938 and 1948. New industrial and commercial buildings, new additions and alterations to old buildings, new dwellings and new public buildings were appraised, together with their contents. As in the 1938 survey, industrial and commercial damages consisted largely of damage to buildings, other structures such as oil and gas tanks, office equipment, stocks and materials, cleaning-up expense, loss of profit and business, loss of wages, and damage to machinery. Whenever possible the amounts of these damages were obtained from the owner or from a responsible representative who was in a position and who had the knowledge to evaluate them. The amounts of these damages, therefore, were on the basis of the dollar value as of 1948. The unit damages per room established for the 1938 survey were doubled to place them on the 1948 basis of values, and then applied to the new dwellings.

Damages to the new development were summed for several flood levels. The curve labeled "New Construction 1938 to 1948" in figure 127 shows the damage in dollars for floods up to 58 feet on the gage. This curve shows that there was a substantial amount of new construction at low stages, amounting to more than the 1938 status at stages below 41 feet.

By combining the 1938 status with the new construction from 1938 to 1948, a third curve was obtained. The damage for the 1938 status was doubled to bring it to the basis of value for 1948, and the new construction was added to this doubled value. The resulting curve labeled "City as of 1948" is shown in figure 127.

1953 appraisal

In 1953 a second supplemental survey of damage at Chattanooga was made. This survey was based on an inspection of aerial photographs which showed a

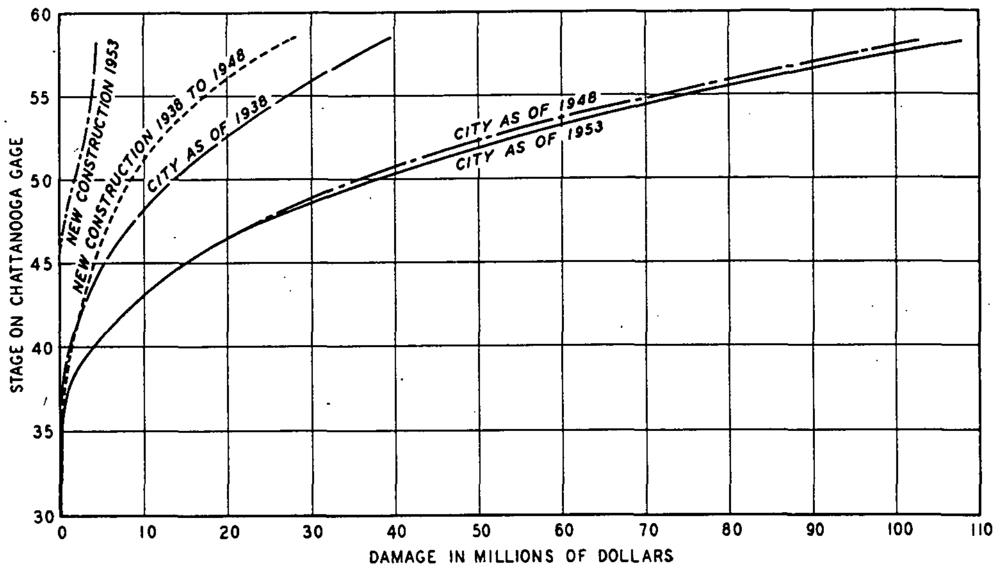


FIGURE 127.—*Flood damage curves—Chattanooga, Tennessee.*

large number of new industrial, commercial, and large-scale housing developments on ground below the natural 1867 flood line. Possible flood damage to these developments was appraised, and a new flood damage curve was prepared. This curve is also shown in figure 127.

The curves in figure 127 give damages for the status of the city in the years 1938, 1948, and 1953. Damage suffered in any actual floods, of course, depends on the status during the occurrence. For example, the actual dollar damage in the 1867 flood (when the population of the city was only about 5,000) was small compared to damage that would have resulted had a flood of the same height occurred in 1953. Also, the actual damage averted in January 1946 ($11.8 million) was greater than for the 1938 status but less than for the 1948 status.

1961 appraisal

A reappraisal of Chattanooga flood damage is scheduled to be completed in 1961.

ANNUAL PREVENTABLE FLOOD DAMAGES

Chattanooga

To determine the average annual preventable flood damage at Chattanooga, stages of all known floods for the 91-year period 1867-1957 were applied to the damage curves for the status of 1938, 1948, and 1953 shown in figure 127. The 119 floods in table 34 are all those which, under natural conditions, reached or exceeded a stage of 30 feet during the 91-

TABLE 34.—*Annual flood damage at Chattanooga for 91-year period, 1867-1957.*

Date of flood	Natural: Crest stage, feet[1]	Natural: Damage in $1,000[3], 1938 status	1948 status	1953 status	Regulated: Crest stage, feet[4]	Regulated: Damage in $1,000, 1938 status	1948 status	1953 status
Mar. 11 1867	57.9	37,700	100,000	105,000	44.0	4,300	12,400	12,500
Feb. 3 1957	54.0[2]	23,900	65,000	66,000	32.24	0	8	20
Mar. 1 1875	53.8	23,298	61,938	64,438	40.6	1,660	5,000	5,000
Apr. 3 1886	52.2	18,800	49,000	52,000	39.1	840	2,700	2,800
Mar. 7 1917	47.7	9,200	24,700	25,500	35.5	24	335	340
Jan. 10 1946	45.8[2]	6,400	17,700	18,000	34.7	9	240	240
Jan. 22 1947	44.5[2]	4,800	13,700	13,800	30.9	0	0	2
Feb. 15 1948	44.3[2]	4,600	13,100	13,200	32.9	1	90	90
Apr. 5 1920	43.6	3,950	11,400	11,400	31.9	0	2	9
Mar. 10 1884	43.5	3,698	10,675	10,550	31.9	0	2	9
Mar. 2 1890	43.2	3,550	10,500	10,300	31.7	0	1	7
Feb. 2 1918	42.7	3.150	9,400	9,200	31.3	0	0	4
Jan. 24 1954	42.0[2]	2,600	7,800	7,600	—	0	0	0
Jan. 2 1902	41.7	2,400	7,200	7,000	30.5	0	0	2
Jan. 19 1882	41.4	2.150	6.600	6,400	30.2	0	0	1

TABLE 34.—*Annual flood damage at Chattanooga for 91-year period, 1867-1957—Continued.*

Date of flood			Natural				Regulated			
			Crest stage, feet[1]	Damage in $1,000[3]			Crest stage, feet[4]	Damage in $1,000		
				1938 status	1948 status	1953 status		1938 status	1948 status	1953 status
Apr.	5	1896	41.4	2,150	6,600	6,400	30.2	0	0	1
Mar.	29	1936	41.3[2]	2,100	6,400	6,200	30.1	0	0	1
Mar.	22	1899	41.0	1,900	5,800	5,700	—	0	0	0
Mar.	11	1891	40.2	1,210	3,675	3,700	—	0	0	0
Mar.	18	1880	39.9	1,250	3,900	3,899	—	0	0	0
Jan.	23	1883	39.9	1,250	3,900	3,900	—	0	0	0
Jan.	15	1879	39.8	1,200	3,700	3,750	—	0	0	0
Feb.	9	1899	39.7	1,100	3,550	3,600	—	0	0	0
Jan.	1	1943	39.7[2]	1,100	3,550	3,600	—	0	0	0
Mar.	14	1897	39.6	931	2,725	2,775	—	0	0	0
Feb.	4	1950	39.6[2]	1,070	3,400	3,450	—	0	0	0
Jan.	17	1892	39.5	1,030	3,250	3,300	—	0	0	0
Mar.	4	1902	39.5	1,030	3,250	3,300	—	0	0	0
Feb.	14	1891	39.1	840	2,700	2,800	—	0	0	0
Apr.	9	1936	38.8[2]	700	2,300	2,400	—	0	0	0
Feb.	11	1884	38.6	610	2,100	2,200	—	0	0	0
Mar.	26	1929	38.5	580	2,000	2,089	—	0	0	0
Dec.	29	1926	38.3	500	1,800	1,850	—	0	0	0
Mar.	30	1944	37.8[2]	330	1,313	1,368	—	0	0	0
Dec.	31	1932	37.5	250	1,100	1,200	—	0	0	0
Feb.	26	1897	37.2	185	900	900	—	0	0	0
Feb.	12	1946	36.8[2]	90	510	533	—	0	0	0
Apr.	10	1892	36.7	105	630	670	—	0	0	0
Dec.	31	1875	36.3	64	500	520	—	0	0	0
Jan.	7	1949	36.3[2]	64	500	520	—	0	0	0
Mar.	23	1875	36.2	28	235	245	—	0	0	0
Mar.	22	1897	35.8	0	0	0	—	0	0	0
Jan.	24	1922	35.8	35	385	400	—	0	0	0
Mar.	23	1955	35.8[2]	35	385	400	—	0	0	0
Feb.	20	1893	35.7	30	360	375	—	0	0	0
Nov.	22	1906	35.7	30	360	375	—	0	0	0
Mar.	30	1913	35.7	27	225	236	—	0	0	0
Dec.	1	1948	35.7	30	360	375	—	0	0	0
Aug.	17	1901	35.6	27	350	355	—	0	0	0
Mar.	30	1951	35.6[2]	27	350	355	—	0	0	0
Jan.	5	1937	35.5[2]	24	335	340	—	0	0	0
May	25	1901	35.3	19	310	310	—	0	0	0
Apr.	26	1883	35.1	15	280	280	—	0	0	0
Jan.	31	1875	34.8	10	250	250	—	0	0	0
Jan.	12	1895	34.7	9	240	240	—	0	0	0
Feb.	13	1921	34.6	8	230	230	—	0	0	0
Feb.	20	1944	34.6[2]	8	230	230	—	0	0	0
Apr.	11	1903	34.5	7	203	203	—	0	0	0
Dec.	21	1915	34.4	6	210	212	—	0	0	0
Mar.	31	1902	34.1	3	139	143	—	0	0	0
Mar.	31	1912	34.1	4	185	191	—	0	0	0
Mar.	28	1917	34.1	2	92	96	—	0	0	0
Mar.	6	1934	34.1	4	185	191	—	0	0	0
Mar.	17	1913	34.0	4	180	185	—	0	0	0
Apr.	18	1956	34.0[2]	4	180	185	—	0	0	0
Mar.	2	1903	33.9	2	132	138	—	0	0	0
Apr.	6	1897	33.6	1	75	78	—	0	0	0
Nov.	9	1885	33.4	2	134	140	—	0	0	0
Jan.	30	1882	33.3	1	31	33	—	0	0	0
Feb.	17	1939	33.3[2]	2	125	130	—	0	0	0
Dec.	28	1914	33.2	1	120	122	—	0	0	0
Feb.	18	1889	33.1	1	110	113	—	0	0	0
Apr.	5	1912	33.1	0	0	0	—	0	0	0
Jan.	1	1916	33.1	1	110	113	—	0	0	0
Mar.	13	1922	33.1	1	100	107	—	0	0	0

TABLE 34.—*Annual flood damage at Chattanooga for 91-year period, 1867-1957—Continued.*

	Natural				Regulated			
		Damage in $1,000[3]				Damage in $1,000		
Date of flood	Crest stage, feet[1]	1938 status	1948 status	1953 status	Crest stage, feet[4]	1938 status	1948 status	1953 status
May 7 1893	33.0	1	100	103	—	0	0	0
Apr. 8 1911	33.0	1	100	103	—	0	0	0
Feb. 19 1903	32.6	1	50	50	—	0	0	0
May 1 1874	32.5	1	30	38	—	0	0	0
Mar. 15 1950	32.5[2]	1	30	38	—	0	0	0
Jan. 22 1936	32.5	1	30	37	—	0	0	0
Jan. 18 1954	32.4[2]	0	17	28	—	0	0	0
Feb. 17 1933	32.4	0	17	28	—	0	0	0
Jan. 5 1919	32.3	0	10	21	—	0	0	0
Feb. 7 1923	32.2	0	7	17	—	0	0	0
Feb. 5 1956	32.2[2]	0	7	17	—	0	0	0
Feb. 25 1891	32.1	0	1	4	—	0	0	0
Apr. 11 1877	32.0	0	3	11	—	0	0	0
Mar. 26 1903	32.0	0	1	2	—	0	0	0
Feb. 24 1953	32.0[2]	0	0	11	—	0	0	0
Dec. 17 1901	31.8	0	1	8	—	0	0	0
Jan. 21 1937	31.8[2]	0	0	2	—	0	0	0
Jan. 14 1901	31.6	0	1	6	—	0	0	0
Mar. 18 1875	31.3	0	0	1	—	0	0	0
Mar. 9 1899	31.3	0	0	3	—	0	0	0
Feb. 6 1936	31.3	0	0	1	—	0	0	0
Jan. 23 1877	30.9	0	0	2	—	0	0	0
Feb. 16 1880	30.9	0	0	2	—	0	0	0
Feb. 28 1887	30.9	0	0	2	—	0	0	0
Feb. 5 1939	30.9[2]	0	0	2	—	0	0	0
Feb. 20 1884	30.8	0	0	0	—	0	0	0
Mar. 31 1888	30.8	0	0	2	—	0	0	0
Mar. 25 1890	30.8	0	0	1	—	0	0	0
Mar. 4 1922	30.8	0	0	0	—	0	0	0
Jul. 2 1928	30.8	0	0	2	—	0	0	0
Apr. 4 1936	30.8[2]	0	0	0	—	0	0	0
Mar. 2 1929	30.7	0	0	2	—	0	0	0
Jan. 10 1936	30.7	0	0	2	—	0	0	0
Dec. 23 1951	30.7[2]	0	0	2	—	0	0	0
Jul. 20 1916	30.4	0	0	1	—	0	0	0
Feb. 26 1927	30.4	0	0	1	—	0	0	0
Feb. 1 1932	30.4	0	0	1	—	0	0	0
Apr. 21 1901	30.3	0	0	1	—	0	0	0
Feb. 12 1937	30.2[2]	0	0	1	—	0	0	0
Jan. 18 1885	30.1	0	0	1	—	0	0	0
May 1 1912	30.1	0	0	1				
Feb. 7 1932	30.1	0	0	0	—	0	0	0
Feb. 29 1944	30.1[2]	0	0	0	—	0	0	0
Apr. 3 1883	30.0	0	0	1	—	0	0	0
Total damage—$1,000		172,249	484,375	498,944		6,834	20,778	21,026
Regulated damage—$1,000		6,834	20,778	21,026				
Preventable damage—$1,000		165,415	463,597	477,918				
Average annual preventable damage—$1,000		1,818	5,094	5,252				

1. All natural crest stages are on the basis of Hales Bar Dam as originally constructed.
2. Computed natural stage.
3. Damages for the later of two successive floods occurring within 7 weeks were adjusted downward to allow for unrepaired damages incurred by the earlier event.
4. For all floods through 1944, regulated crest stages are based on a one-third reduction of natural crest flows and Hales Bar Dam as constructed in 1948. From 1945 to 1948 observed crest stages have been corrected for the change in Hales Bar Dam. All later crests are as observed. Regulated crests below 30 feet are not shown.

year period. Natural conditions are defined as those prevailing after the original construction of Hales Bar Dam, completed in 1913. Earlier observed stages were adjusted upward to allow for the influence of Hales Bar Dam and to make them comparable with conditions existing at the time TVA was created. Since 1936, when TVA flood regulation began, computed natural stages applying to the 1913 Hales Bar status were used.

Damages for the later of two successive floods occurring within seven weeks were reduced to allow for unrepaired damages of the earlier flood.

On the basis of the above assumptions, the difference between the average annual flood damage at Chattanooga under pre-TVA conditions and under post-TVA conditions is $5,252,000 for the status of 1953.

Damage actually prevented at Chattanooga in 25 years since Norris Dam was closed in 1936 amounts to $120,213,000 (table 35) or an average of $4,810,000 per year. This average amount is somewhat less than that obtained from consideration of the period of gage record of 91 years (1867-1957) at 1953 status. Figure 128 shows graphically the damages prevented each year at Chattanooga since Norris Dam was closed in 1936. The damages listed in table 35 are intended to represent the damage actually averted as of the date of the flood. Damages for floods prior to 1946 were taken from the 1938 damage curve in figure 127. For the floods in 1946 damage from the 1938 curve was increased by 50 percent to allow for the change in the dollar value between 1938 and 1946, and by 50 percent of the new development found in the 1948 survey. For the 1947 flood, the 1938 value was increased by 75 percent, and by 75 percent of the new construction found in 1948. For floods from 1948 to 1951, the 1948 damage curve was used, and for floods from 1954 to 1958 the 1953 damage curve was used.

TABLE 35.—*Flood damages averted at Chattanooga for period 1936-1960.*

Date of actual flood			Flood damages: From actual flood	Flood damages: From natural flood	Damages averted[1]
Mar.	29	1936	$170,000	$ 2,100,000	$ 1,930,000
Apr.	9	1936	21,000	700,000	679,000
Jan.	4	1937	5,000	24,000	19,000
Dec.	30	1942	35,000	1,100,000	1,065,000
Feb.	19	1944	0	8,000	8,000
Mar.	30	1944	5,000	330,000	325,000
Jan.	9	1946	200,000	12,000,000	11,800,000
Feb.	11	1946	0	415,000	415,000
Jan.	21	1947	5,000	11,500,000	11,495,000
Feb.	14	1948	160,000	13,100,000	12,940,000
Nov.	29	1948	0	360,000	360,000
Jan.	6	1949	0	500,000	500,000
Feb.	2	1950	0	3,400,000	3,400,000
Mar.	15	1950	0	30,000	30,000
Mar.	30	1951	0	350,000	350,000
Jan.	22	1954	0	7,600,000	7,600,000
Mar.	22	1955	0	400,000	400,000
Feb.	4	1956	0	17,000	17,000
Apr.	17	1956	0	185,000	185,000
Feb.	2	1957	20,000	66,000,000	65,980,000
Apr.	5	1957	0	2,000	2,000
Nov.	20	1957	0	710,000	710,000
Apr.	30	1958	0	—[2]	0
May	10	1958	0	3,000	3,000
Total			$621,000	$120,834,000	$120,213,000

1. Status of date of flood—1953 status used since that date.
2. Included in following flood.

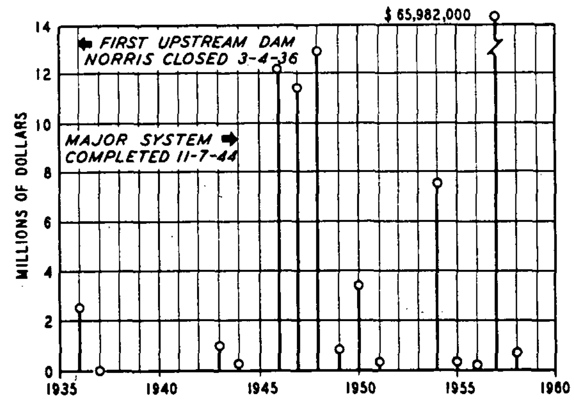


FIGURE 128.—*Flood damages prevented at Chattanooga—1936-1960 fiscal years.*

Other valley towns

Estimates of damage preventable by the TVA reservoir system have been made for other towns along the Tennessee River and its tributaries. Since the flood damages at these towns are much less than at Chattanooga, detailed damage surveys were not believed warranted. In general, a count of houses and other buildings subject to flooding was made and a unit value was assigned which depended on the depth of flooding. These unit values represented the dollar value as of 1953-1954 and the estimated damages are, therefore, comparable with the damages determined for Chattanooga for 1953 status. For Dayton, Clinton, Elizabethton, and Kingsport, field surveys of the property affected were made, but in the case of Knoxville, Loudon, and Lenoir City the number of buildings affected was determined from maps. Then, using 1953 status with no reservoir protection as a basis, tables 36, 37, and 38 were compiled. Table 36 shows flood damages at Knoxville, Lenoir City, Loudon, and Dayton, table 37 at Clinton and table 38 at Elizabethton and Kingsport for the period of record shown; the average annual damage is also given.

Several other towns such as Kingston, Soddy, Jasper, South Pittsburg, Bridgeport, Decatur, and Florence will have smaller flood damage due to the TVA reservoir system, but no separate determinations have been made of annual flood damage at these places. In addition, damages to many industries located outside the cities studied will be prevented or reduced. An estimate of $10,000 annual damage was considered reasonable for this group.

TABLE 36.—*Flood damages*[1] *at Knoxville, Lenoir City, Loudon, and Dayton.*

Year of Flood	Knoxville		Lenoir City		Loudon		Dayton	
	Elev.	Damage	Elev.	Damage	Elev.	Damage	Elev.	Damage
1867	842.7	$1,850,000	785.9	$ 775,000	774.1	$ 600,000	702	$ 930,000
1875	840.9	1,490,000	783.0	585,000	770.9	450,000	698	500,000
1886	828.7	169,000	773.2	195,000	764.4	215,000	695	180,000
1890	—	—	762.8	25,000	753.0	3,000	—	—
1891	—	—	761.0	6,000	751.4	1,000	—	—
1892	—	—	763.9	37,000	754.0	5,000	—	—
1896	829.7	216,000	765.5	56,000	755.6	16,000	—	—
1897	824.5	38,000	761.0	6,000	751.4	1,000	—	—
1899	826.9	104,000	—	—	—	—	—	—
1901	830.8	283,000	—	—	—	—	—	—
1901	828.7	169,000	—	—	—	—	—	—
1902	832.1	380,000	—	—	—	—	—	—
1906	—	—	765.2	53,000	755.2	13,000	—	—
1916	829.4	200,000	762.8	25,000	753.0	3,000	—	—
1917	828.7	169,000	768.7	102,000	758.8	70,000	—	—
1918	823.0	13,000	762.2	18,000	752.5	2,000	—	—
1920	826.1	78,000	767.6	85,000	757.6	46,000	—	—
1932	—	—	760.6	2,000	751.0	1,000	—	—
1936	—	—	762.3	20,000	752.2	2,000	—	—
1942	823.3	18,000	—	—	—	—	—	—
1946	826.6	94,000	764.4	43,000	754.4	7,000	—	—
1946	822.6	8,000	760.3	0	750.9	1,000	—	—
1947	824.8	43,000	765.2	53,000	755.2	13,000	—	—
1948	—	—	760.4	1,000	750.9	1,000	—	—
1954	826.1	78,000	—	—	—	—	—	—
1956	822.7	9,000	—	—	—	—	—	—
1957	833.3	465,000	775.4	260,000	764.9	230,000	697	390,000
Total		$5,874,000		$2,347,000		$1,680,000		$2,000,000
Length of record		91		91		91		91
Average annual flood damage		$65,000		$26,000		$18,000		$22,000

1. 1953 status with no reservoir protection—practically no damage would be shown if the computations had assumed TVA reservoirs in operation.

Agricultural land

Between the tributary dams and the Tennessee River and in the Tennessee River reservoirs above the normal pool levels, there is a large area of agricultural land which is given flood protection. This protection is of most value during the crop season, which extends approximately for seven months from May through November.

Agricultural damage preventable by the TVA reservoir system was determined for eight main Tennessee River reservoirs (Wilson was excluded) and for part of the five major tributaries below the large storage reservoirs. The general method was to (1) select for each of these locations the flow at which agricultural damage begins; (2) count the number of natural floods in each of several flow bands exceeding this flow in (a) May and in (b) the period of June through November (the number of floods was decreased in several cases to allow for floods which, on the basis of experience, probably would not be reduced to the non-damaging flow); (3) determine the area that would be protected in an average flood in each of the various flow bands; (4) multiply the area protected in each band by the net number of floods in that band and sum these results; (5) multiply the total area protected in all the floods by the unit damage values of $10 per acre for floods in May and $30 per acre for floods from June through November; and (6) divide the total damage by the number of years of flood record.

Table 39 shows the number of floods and the damages preventable at each location in the two periods considered, the number of years in the flood record, and the average annual preventable damages which total $363,000. The unit damage value of $10 per acre in May is equivalent to an average loss of one-quarter of a corn crop on the basis of a yield of 40 bushels per acre at $1 per bushel. The value of $30 per acre is equivalent to an average loss of three-quarters of a corn crop for the same yield and value per bushel.

TABLE 37.—*Flood damages at Clinton.*[1]

Year of Flood	Elev.	Damage	Year of Flood	Elev.	Damage
1884	808.5	$ 21,000	1918	816.5	$ 270,000
1886	818.0	400,000	1919	808.0	18,000
1887	804.0	4,000	1920	805.6	8,000
1889	803.2	2,000	1920	803.8	4,000
1890	812.1	64,000	1923	809.2	25,000
1891	807.7	16,000	1926	809.0	24,000
1891	803.7	3,000	1927	802.1	1,000
1892	804.8	6,000	1928	805.1	7,000
1893	809.1	25,000	1929	811.0	45,000
1895	802.5	1,000	1929	804.8	6,000
1896	814.3	140,000	1932	808.7	22,000
1897	815.5	205,000	1932	802.0	1,000
1899	804.8	6,000	1934	802.9	2,000
1899	804.2	5,000	1935	804.0	4,000
1901	802.9	2,000	1936	803.1	2,000
1901	803.5	3,000	1937	803.5	3,000
1901	807.5	15,000	1937	805.1	7,000
1902	809.0	24,000	1939	807.6	16,000
1902	806.6	12,000	1942	803.5	3,000
1903	803.2	2,000	1944	805.8	9,000
1906	807.5	15,000	1946	811.9	60,000
1907	802.5	1,000	1947	804.1	4,000
1911	802.5	1,000	1948	807.7	16,000
1911	802.1	1,000	1949	802.6	2,000
1912	808.5	21,000	1950	810.0	33,000
1912	802.1	1,000	1950	801.9	1,000
1913	806.7	12,000	1951	805.9	9,000
1915	807.1	13,000	1953	801.8	1,000
1917	808.0	18,000	1955	806.5	11,000
1917	714.5	150,000	1956	805.6	8,000
1917	814.5	150,000	1957	813.2	95,000
(Continued)			Total		$1,906,000
			Length of record		74
			Average annual flood damage		$ 26,000

1. 1953 status with no reservoir protection—practically no damage would be shown if the computations had assumed TVA reservoirs in operation.

TABLE 38.—*Flood damages at Elizabethton and Kingsport.*[1]

Elizabethton, Tennessee			Kingsport, Tennessee		
Flood	Elev.	Damage	Flood	Elev.	Damage
1867	1504	$ 960,000	1847	1201	$ 9,000
1886	1505	1,170,000	1861	1201	3,000
1896	1502	170,000	1862	1202	45,000
1897	1503	780,000	1867	1207	1,540,000
1900	1502	170,000	1875	1204	540,000
1901	1507.7	1,820,000	1886	1204	540,000
1901	1501	50,000	1896	1203	100,000
1902	1506.2	1,430,000	1897	1203	100,000
1916	1501.4	90,000	1901	1207	1,750,000
1940	1507.1	1,650,000	1901	1202	20,000
			1902	1203	195,000
Total		$8,290,000	1906	1201	3,000
			1916	1201	3,000
Length of record		90 years	1940	1203.1	140,000
			Total		$4,988,000
Average annual flood damage		$ 92,000	Length of record		110 years
			Average annual flood damage		$ 45,000

1. 1953 status with no reservoir protection—practically no damage would be shown if the computations had assumed TVA reservoirs in operation.

Lower Ohio and Mississippi Rivers

Kentucky Reservoir was built primarily for the reduction of flood crests on the lower Ohio and Mississippi Rivers. Storage reservations in Pickwick, Wheeler, and Guntersville Reservoirs also were provided largely to aid in reducing floods in those rivers. Although reservoirs above Chattanooga, both tributary and main-river, are operated with Chattanooga as the focal point, they may at times aid greatly in lowering Ohio and Mississippi River floods. In addition to retaining large volumes during Tennessee

TABLE 39.—*Annual flood damages to agricultural land preventable by TVA reservoirs—Tennessee River Basin.*

Location	May floods		June to November floods				
	No.	Damages preventable at $10 per acre	No.	Damages preventable at $30 per acre	Total preventable damages	Length of record, years	Annual preventable damages
Holston River	19	$ 134,190	41	$ 689,700	$ 823,890	48	$ 17,000
French Broad River	27	142,020	72	1,497,600	1,639,620	51	32,000
Little Tenn. River	20	53,190	32	620,190	673,380	40	17,000
Clinch River	33	223,110	64	1,133,670	1,356,780	51	27,000
Hiwassee River	23	111,560	49	740,460	852,020	40	21,000
Ft. Loudoun Reservoir	4	10,140	10	33,900	44,040	42	10,000
Watts Bar Reservoir	6	3,570	14	103,800	107,370	38	3,000
Chickamauga Reservoir	13	204,320	21	640,620	844,940	62	14,000
Hales Bar Reservoir	13	59,830	21	212,100	271,930	62	4,000
Guntersville Reservoir	13	730,920	27	3,501,000	4,231,920	62	68,000
Wheeler Reservoir	5	215,060	8	1,278,240	1,493,300	52	29,000
Pickwick Reservoir	3	23,310	2	109,980	133,290	64	2,000
Kentucky Reservoir	12	1,021,270	14	6,595,230	7,616,500	64	119,000
				Total average annual preventable damage			$363,000

River floods, these upper-Basin reservoirs will materially lower Ohio and Mississippi River floods as a result of their normal filling during April and May.

There are approximately 4,000,000 acres of land on the lower Ohio and Mississippi Rivers not protected by levees but benefited by operation of the TVA reservoir system, principally Kentucky Reservoir. Damage preventable on this land includes that to railroads, highways, crops, and other property.

The computation of average annual damages preventable on this land was based on the flood record of 101 years at Cairo, Illinois, extending from 1857 to 1957, with the historic flood of 1844 added. All early flood crests were adjusted to bring them to present-day channel conditions.

For selecting floods occurring before closure of Kentucky Dam in August 1944, the criterion for regulation was when the Cairo crest stage exceeded 54 feet between January 1 and March 31, 54 feet on April 1 to 44 feet on April 30, 44 feet between May 1 and November 30, and 44 feet on December 1 to 54 feet on December 31. For those floods after 1944 the criterion was a stage of 40 feet, the point at which damage begins.

For all floods from 1903 to 1944 and that of 1897, flood crest reductions used to determine average annual damage were those computed for an assumed reservoir operation. Flood crest reductions for floods since 1944 were the actual reductions computed by standard routing procedures. All other floods, those prior to 1903 except 1897, were assumed to be reduced 1.7 feet, the average of reductions determined in actual operations since closure of Kentucky Dam in 1944 and assumed operations between 1903 and 1944, but no floods were assumed to be reduced below the 44-foot stage. The unregulated crest stage for present-day channel conditions and the reduction in feet are given in table 40.

TABLE 40.—*Preventable damages along the Mississippi River by TVA reservoirs in all floods exceeding criterion[1] stage at Cairo—1844, 1858-1957 (101 years).*

Year	Date	Unregulated crest stage,[2] feet	Reduction in feet	Preventable damage,[3] $1000's
1844 (historic)	July 15	52.2	1.7	$6,755.0
1858	April 26	49.9	1.7	9,040.0
	June 21	55.7	1.7	
1859	May 7	52.2	1.7	5,285.0
1861	April 24	47.5	1.7	5,415.0
	May 18	47.5	1.7	
1862	May 2	57.3	1.7	7,390.0
1865	April 24	52.7	1.7	3,926.0
1866	May 1	46.9	1.7	3,852.0
1867	March 21	57.5	1.7	3,632.0
1868	April 23	47.0	1.7	5,712.0
	May 19	50.7	1.7	
1874	April 26	53.0	1.7	4,555.0
1875	July 23	45.3	1.3	752.0
	August 8	50.0	1.7	
1876	May 14	46.4	1.7	4,803.0
1881	April 20	50.9	1.7	4,624.5
	May 9	47.0	1.7	
1882	Feb. 26	58.1	1.7	9,075.0
	May 23	46.8	1.7	
	June 6	46.0	1.7	
1883	Feb. 27	58.3	1.7	4,308.0
	April 27	45.8	1.7	
1884	Feb. 23	58.0	1.7	5,470.0
	March 30	54.7	1.7	
1886	April 19	57.5	1.7	5,800.0
1887	March 9	54.6	1.7	1,979.0
1890	March 12	55.0	1.7	3,903.0
	April 6	54.8	1.7	
1892	April 28	54.3	1.7	7,355.0
	May 25	51.1	1.7	
1893	April 23	48.4	1.7	8,990.0
	May 9	55.8	1.7	
	June 6	47.7	1.7	
1897	March 25	58.1	1.8	4,709.8
	April 15	55.7	0.4	
1898	April 6	56.3	1.7	4,050.0
1901	May 2	47.6	1.7	3,994.0
1903	March 16	57.2	3.1	14,720.0
	April 23	50.2	3.0	
	June 13	47.9	2.4	
1904	April 5	55.4	2.2	4,740.0
	May 6	46.1	0.7	
1906	April 9	52.5	1.2	2,002.0
1907	Jan. 27	56.9	0.1	177.0
1908	April 13	50.3	0.6	3,406.0
	May 20	50.0	1.3	
1909	May 13	46.7	2.5	7,272.0
	July 18	47.9	2.2	
1911	April 21	50.4	4.6	8,238.0
1912	April 6	59.3	1.3	10,820.0
	May 4	55.6	3.8	
1913	Jan. 29	55.1	1.1	3,363.0
	April 7	59.6	0.6	
1916	Feb. 4	57.9	0.8	780.0

TABLE 40.—*Preventable damages along the Mississippi River by TVA reservoirs in all floods exceeding criterion[1] stage at Cairo—1844, 1858-1957 (101 years)—Continued.*

Year	Date	Unregulated crest stage,[2] feet	Reduction in feet	Preventable damage,[3] $1000's
1917	April 4	53.5	3.9	8,845.0
	June 16	47.7	1.2	
1919	March 24	54.4	0.4	4,620.0
	May 16	45.4	1.7	
1920	May 1	51.7	3.1	5,779.0
	May 23	45.4	2.0	
1922	March 25	55.0	1.0	4,336.0
	April 25	54.9	1.1	
1926	Oct. 12	44.5	0.5	63.8
1927	April 19	57.8	1.2	7,010.0
	June 8	49.6	1.8	
1928	April 28	46.5	2.5	5,823.0
	July 6	50.6	1.5	
1929	March 20	56.2	2.2	14,215.0
	April 5	55.8	1.8	
	May 19	56.9	2.1	
1933	May 21	51.8	2.1	5,770.0
1935	May 26	45.9	1.8	5,765.0
1936	April 16	52.7	2.7	5,320.0
1937	Feb. 4	59.5	1.2	11,885.0
	May 9	48.6	3.3	
1939	April 25	50.4	0.8	1,483.0
1940	May 3	44.5	1.4	3,099.6
1943	May 30	53.0	0.4	1,736.0
1944	April 29	51.2	0.9	2,287.0
1945	March 13	55.4	1.5	2,454.0
	March 23	54.3	0.4	
	April 3, 4	54.3	0.6	
1946	Jan. 18	53.5	1.4	1,163.0
	Feb. 23	46.3	0.4	
1947	Jan. 29	42.9	1.9	1,507.0
	April 18	48.0	0.9	
1948	Feb. 24	48.7	1.9	3,925.0
	March 6	46.1	0.5	
	April 4	53.4	1.8	
	April 21	49.0	1.1	
1949	Feb. 1	51.3	0.6	718.7
	Feb. 27	50.5	1.2	
	April 6	46.9	0.1	
1950	Jan. 20	57.2	1.8	3,420.0
	Feb. 17	57.1	1.2	
	April 10	47.3	0.5	
1951	Feb. 14	41.5	0.2	488.3
	Feb. 27	49.0	0	
	April 9	47.5	0	
	April 24	46.7	0.5	
1952	March 28	51.2	0.5	535.5
1955	March 28	51.3	1.2	1,270.0
1956	Feb. 29	45.8	2.1	655.0
	March 24	42.3	1.6	
1957	Feb. 15	47.2	1.5	4,659.0
	April 17	46.8	3.0	
Total preventable damages $1,000's				$289,726.2
Average annual preventable damages $1,000's				$2,869.0

1. 54 feet Jan. 1 to March 31—54 feet April 1 to 44 feet April 30—44 feet May 1 to Nov. 30—44 feet Dec. 1 to 54 feet Dec. 31. Since closure of Kentucky Dam in 1944 the table includes those floods above 40 feet for which natural crests are available.
2. All flood stages are for present-day channel conditions.
3. 1953 status.

Preventable damages were determined from curves (fig. 129) prepared in 1944 from data in the report entitled "Value of Flood Height Reduction from TVA Reservoirs to the Alluvial Valley of the Lower Mississippi River," (House Document No. 455, 76th Congress, 1st Session). The curves show the damages prevented in Mississippi River floods occurring at various dates in the year in terms of the value of 1 foot of reduction in Cairo crest stage. The curves are entered with the average of the natural and regulated crest stages and the average date between the date of the natural crest and the date the natural flood would have receded to the stage of the regulated crest. The curves represent monetary values of 1936-1937. The values are based on property values and average yields and market values of river bottom corn and cotton as used in the aforementioned report.

Because the preventable damage values for different floods occurring in the same season are closely related, a single monetary value was determined for all floods occurring in that season. A schedule was established, depending on the proximity of multiple crests, which adjusted the estimated damages as realistically as possible.

To bring the values to the basis of 1953, as was done for the damages preventable at Chattanooga and other cities in the Tennessee River Basin, adjustments were made to the 1936-1937 values on the basis of building cost and agricultural price indexes. Damage to property other than crops—as represented by the values where the curves are level in January and February—was multiplied by 2.50 (derived from the building cost index), and crop damage—as represented by the difference between the January-February damage and the damage shown by the

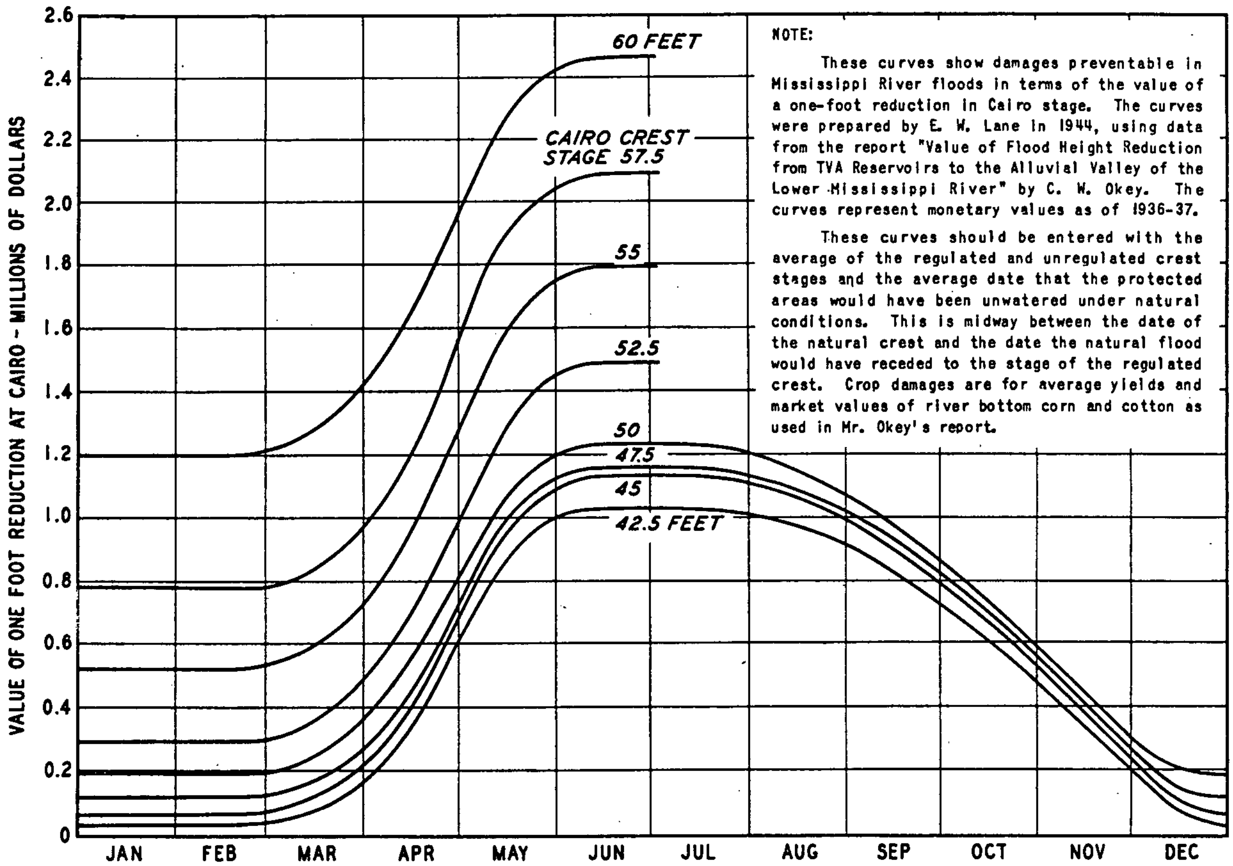


FIGURE 129.—*Flood damage curves—lower Ohio and Mississippi Rivers.*

curves at other dates—was multiplied by 3.0 (derived from the agricultural price index).

Table 40 lists the preventable damages for all years from 1857 to 1957, plus the historic flood of 1844. On the basis of the foregoing assumptions, the total preventable damages for the 101 years for channel conditions and property status of 1953 are $289,726,000 and the average annual preventable damages are $2,869,000.

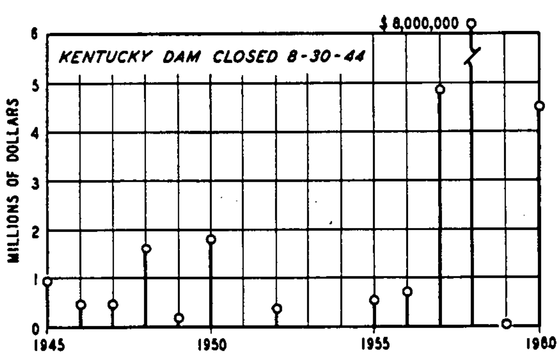


FIGURE 130.—*Flood damages prevented along lower Ohio and Mississippi Rivers—1945-1960.*

Damages actually prevented on the lower Ohio and Mississippi Rivers in the 16 years after Kentucky Dam was completed in 1945 amount to $24,651,000, assuming status of date of flood, or an average of

TABLE 41.—*Flood damages averted on the lower Ohio and Mississippi Rivers by storage in TVA reservoirs—1945-1960.*

Year	Damages averted[1]
1945	$ 970,000
1946	500,000
1947	480,000
1948	1,600,000
1949	200,000
1950	1,800,000
1951	0
1952	400,000
1953	0
1954	0
1955	580,000
1956	696,000
1957	4,875,000
1958	8,000,000
1959	50,000
1960	4,500,000
	$24,651,000

1. Status of date of flood

about $1,540,000 per year (table 41). This average amount is substantially less than that obtained from consideration of the period of gage record of 101 years at 1953 status. Figure 130 shows graphically the damages prevented each year along the lower Ohio and Mississippi Rivers since Kentucky Dam was closed in 1944.

SUMMARY OF ANNUAL FLOOD DAMAGES

The principal purpose of determining flood damages is for estimating the monetary value of benefits which may be weighed against the cost in calculating the economic feasibility of flood prevention. It may be questioned whether or not to include damage to the increased development of a city after completion of the flood protection works as part of the benefits assignable to those works. Also, since the construction of the TVA flood control system has extended over a period of some 24 years, it is difficult to establish a definite date after which any increased development should be excluded from flood control benefits.

Although flood crest reductions have been obtained since 1936 when Norris Reservoir was placed in operation, it was not until 1952 that the present system was completed. Moreover, during World War II there was a substantial amount of industrial development on land subject to flooding which would have taken place regardless of whether or not flood control was provided. The adoption of a date near the end of the construction period seems the most reasonable for the determination of annual flood damage, and since it was convenient to make a resurvey of the city of Chattanooga in 1953, that year was selected. The status of development and dollar value of 1953 is the basis for annual flood damage at Chattanooga. Because field surveys of development in other towns and cities as of 1953 were not justified, the development shown on maps was used, but the unit Chattanooga values for each building were used and therefore the results are on the basis of the 1953 dollar value.

Table 42 shows the total average annual flood damages in the Tennessee River Basin and the lower Ohio and Mississippi Basins assuming that all floods of record occurred with the status of 1953. The table includes only those flood damages preventable by the TVA multiple-purpose reservoir system. It does not include flood damages on the tributaries where there are no reservoirs, as the Duck and Elk Rivers, nor on tributaries upstream from storage reservoirs, such as on the upper French Broad River above Douglas Reservoir. Damage at these other locations will not be affected by the present reservoir system. Damage in the areas now covered by reservoirs for which easements or fee simple title have been acquired was also excluded from the total amounts given in table 42.

TABLE 42.—*Summary of average annual flood damages preventable by the TVA reservoir system.*

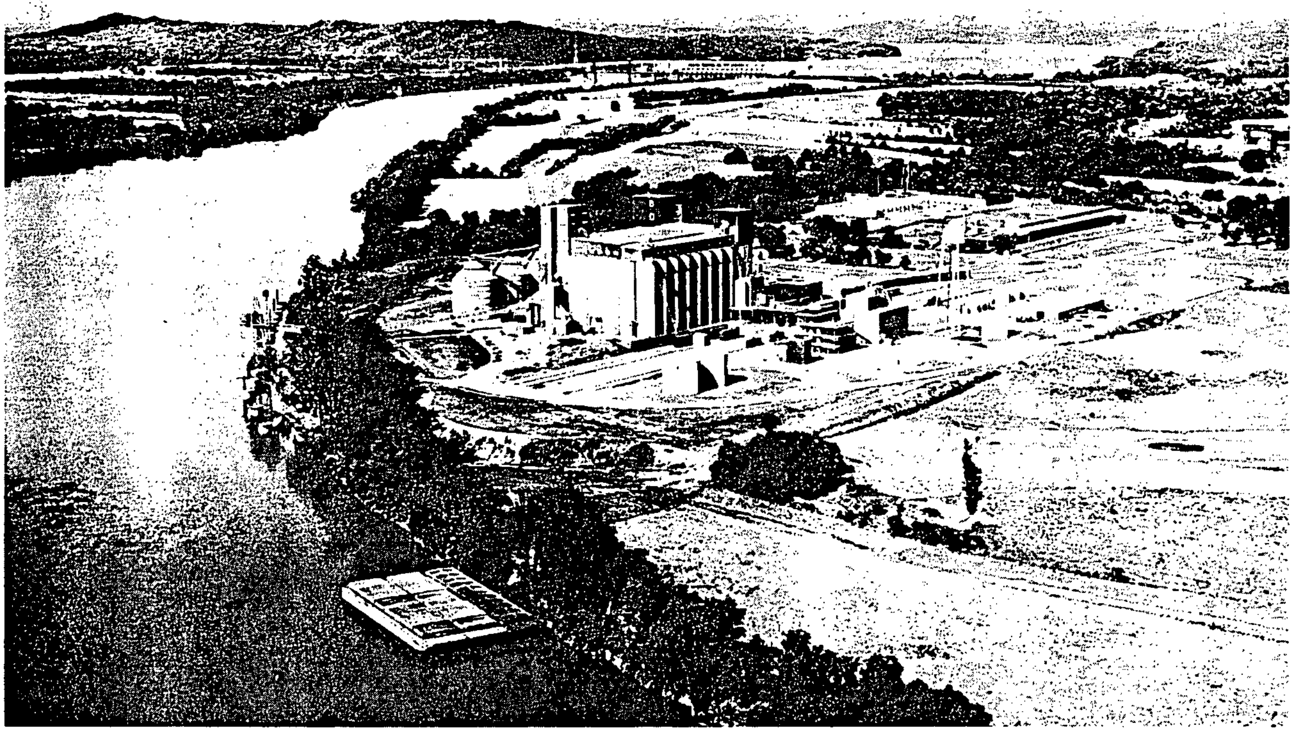
Location	Average annual flood damages
Tennessee Basin	
Urban and industrial properties:	
Chattanooga	$5,252,000
Knoxville	65,000
Lenoir City	26,000
Loudoun	18,000
Dayton	22,000
Clinton	26,000
Kingsport	45,000
Elizabethton	92,000
Other	10,000
Total urban and industrial properties	5,556,000
Agricultural lands	363,000
Total Tennessee Basin	5,919,000
Lower Ohio and Mississippi Basins	2,869,000
Grand total	$8,788,000

DAMAGE IN FUTURE FLOODS

Estimates of average annual flood damages were based on flood heights already experienced, no consideration being given to possible higher floods nor to possible increased dollar value of property.

It is difficult to determine the damages which would be suffered at stages higher than the maximum known flood. In addition to greater direct property destruction, a flood equal to the maximum probable flood would so greatly disrupt the entire business life of a community that it would be several months or even years before normal conditions were reestablished.

Field examinations of additional areas subject to flooding by the maximum probable flood have not been made, but in the previously mentioned report, "The Chattanooga Flood Control Problem," a computation shows that the damage to Chattanooga for a flood 10 feet higher than the maximum known flood would be $70,000,000 (1938 status). If the same relative increase between 1938 and 1953 is assumed for the 10-foot higher stage as was indicated for the stage of the maximum known flood, the 1953 damage would be nearly $200,000,000. Even with regulation by TVA reservoirs, but without local protection works, the maximum probable flood would cause damage of more than $100,000,000 (1953 status).



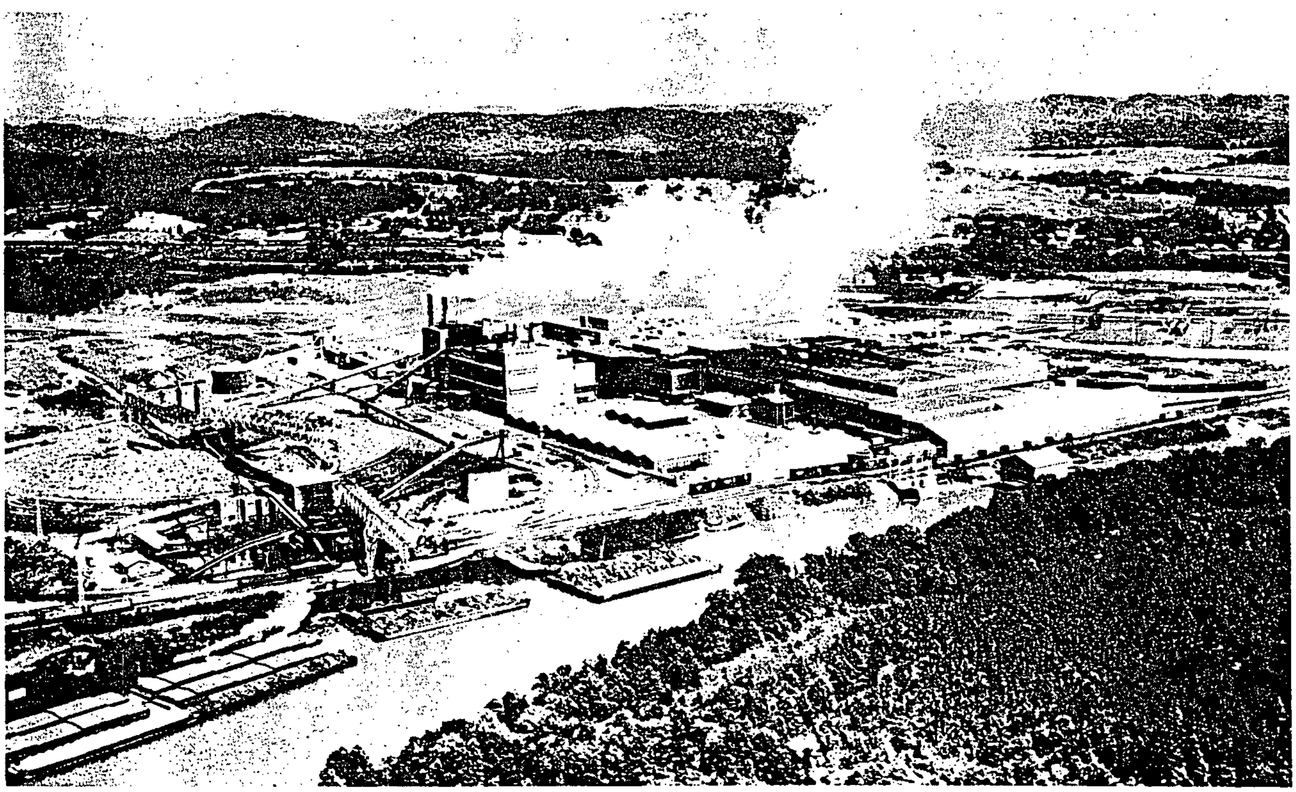


FIGURE 131.—*New industries in areas protected from floods are among the many benefits accruing from flood control. Top view: Central Soya Co. plant below Chickamauga Dam near Chattanooga. Bottom view: Bowaters paper mill on the Hiwassee River arm of the Chickamauga Reservoir.*

CHAPTER 13

BENEFITS FROM FLOOD CONTROL

Whereas the preceding chapter concerns damage from floods, this chapter concerns benefits from flood control. It relates damages to benefits, classifies benefits, and then discusses the many factors affecting benefit calculation. The remainder of the chapter discusses actual benefits — resulting from TVA's flood control system—in the Tennessee Valley and below Kentucky Dam on the Tennessee, Ohio, and Mississippi Rivers, and also incidental benefits to other water use programs. A summary of the average annual flood control benefits ends the chapter.

RELATION OF DAMAGES TO BENEFITS

The determination of damages from floods and the conversion of these damages into benefits may be rather elusive. The damages can be determined from the stages of past floods, but the question immediately arises as to whether these stages should be applied to the stage of development of the property at the time of the flood, to the current state of development of the property, or to a predicted state of development at some time in the future. The damage picture and, consequently, the annual benefits will be entirely different if a great flood should occur next year or 100 years hence. If, in addition, it is proposed to reduce projected future benefits to present worth to compare them with the investment in the project, the problem becomes largely an academic exercise because of the personal opinions and speculations involved.

The determination of benefits as discussed herein is primarily for the purpose of finding out whether the cost of the contemplated flood control or flood protection project is justified. In general, the benefits are assumed to be directly related to the damages averted by the flood control works. Also, in a flood control district financed by local taxation, the assessment levied on property within the district usually is in proportion to the primary benefit received.

CLASSIFICATION OF BENEFITS

Various agencies have used a wide assortment of terms in classifying benefits. A number of Federal agencies have attempted to codify these terms, and the terminology suggested by them seems to be simple and workable. According to this classification, the term "tangible benefits" is used for all benefits to which a definite monetary value can be assigned, while "intangible benefits" include those to which no definite monetary value can be assigned.

Tangible benefits are broken down into "primary" and "secondary." The former includes "direct" benefits which are related to physical damage to property, crops, and land that would be averted with protection from floods. In addition to avoidance of physical damage, primary benefits may also include "indirect" benefits, such as the avoidance of the loss of wages; loss of industrial output; loss of retail sales profit; loss of public utilities income; the costs of evacuation and reoccupation of flooded areas; cost of emergency protection and flood fighting; and cost of relief, care, and rehabilitation of flood victims.

Secondary benefits include such items as increase in property values, new industries, and new payrolls to be expected. However, it may be difficult to evaluate these, particularly with relation to the life of the project.

Intangible benefits are not often used because of the difficulty of assigning dependable monetary values to them. They include the prevention of loss of life or of impairment of health, removal of industries, preservation of scenic resources and historical sites, and inconveniences to the public.

FACTORS AFFECTING CALCULATION OF BENEFITS

Average annual benefits

The most generally accepted method of calculating the benefit-cost relationship is the comparison of average annual, primary, tangible benefits with average annual cost, omitting such refinements as present worth, prediction of future construction costs, change in state of development of the area to be protected, and other uncertain elements of like nature.

Since the order of construction of projects and the scope of development in the TVA system were determined by the requirements of the TVA Act, not a great deal of effort was spent in calculating benefits to the last dollar so long as the primary benefits more than justified the cost allocated to flood control in a given project.

Average annual costs

The cost of a proposed project should not be difficult to determine provided the requisite physical data have been obtained at the site for preparation of an adequate plan for construction. On the other hand, conversion of capital cost to an average annual cost basis for use in the benefit-cost analysis requires a great deal of judgment. TVA's approach to this problem is discussed in the following chapter.

Increase in value of property protected

It is generally true that in time there will be an increase in value of the property to be protected from flood damage. However, it is uncertain whether this increase will occur in time to materially affect the average annual benefit due to protection. It is also much more difficult to make allowance for increase of value for flood control in a multiple-purpose project than in a single-purpose project, since other purposes should share in the credit for increase in value. In the past it has been the policy of Federal agencies not to include estimates of benefits due to increases in value of property protected when the benefits are purely local in nature and the protective works are of entirely local significance. When the benefits are of a regional nature, however, as in the case of the lower Ohio and Mississippi Rivers, it is proper to include increased property values among the benefits.

Potential future benefits

Potential future benefits would, with few exceptions, fall in the class of "secondary" benefits. TVA, in planning its present multiple-purpose system, considered them too speculative to use in preparing reports on flood control in connection with multiple-purpose projects.

Differences between urban and rural areas

Flood damages are obviously much greater in the highly developed, densely populated urban areas vulnerable to all floods, than in the rural areas ordinarily subject to flood damage only during the crop season.

The term "urban area" connotes an area largely occupied by dwellings, commercial buildings and industrial plants. Level land at relatively low elevation, such as the river bottom land, has been at a premium as these areas developed and resulting pressures have forced buildings to lower and lower elevations until they are within the path of relatively low floods. On the other hand, outside the urban areas buildings of a type that would suffer serious damage generally have not been constructed below high water elevations. Corn cribs, hay barns, and tobacco sheds are about the only buildings found on the flood plains in rural areas. Residences usually occupy higher, adjacent ground.

Damaging floods which occur during the crop season are largely confined to the extreme eastern and southeastern sections of the Valley—see figure 73, page 110, showing distribution of floods at Kingsport, Tennessee. They are usually caused by tropical hurricanes which occasionally drop heavy precipitation on limited areas west of the Blue Ridge.

In general, it may be said of the Tennessee Valley, with the exception of a few areas, that floods are confined to the season before crops are planted and after they have been harvested. Accordingly, the damage to farming operations is rather limited. On the other hand, the urban areas do not stop operation during the flood season, and the fact that excessive floods do not occur during the summer does not relieve them from the ever-present menace of severe damage during the winter and spring months. The alleviation of these damages is productive of at least equal benefits.

Present worth of future damage

A perusal of any long-time record of floods will show that they do not occur in the same sequence in any two periods that may be selected for study. This, of course, strongly indicates that future floods will not follow the same pattern as in the past. Except for the 1867 flood, that of 1882 on the Ohio River at Paducah was the highest known up to that time. However, higher floods than that of 1882 occurred in 1883 and 1884. Again in 1912 there was a flood that equalled that of 1882, and in 1913 a flood as high as that of 1884. In 1937 there was a stage at Paducah that exceeded those of 1884 and 1913 by over 6 feet; and although some of this increase may have been due to confinement by levees, this effect is relatively minor. This illustrates the impossibility of predicting the time at which high floods will occur in the future or the sequence in which they will occur. Without this knowledge, attempts to predict future damages within the life of the project or to discount them to present worth are simply guesses. It is felt that as good an estimate of benefits as it is possible to make is to take the past record of floods as they occurred and estimate the damage on the basis of the state of development as of the time of construction of the protective works. Admittedly, this cannot be exact, but the inaccuracies will only be compounded by adding assumptions as to future events.

1953 basis for benefits

The construction of the existing TVA flood control system extended over a period of nearly 20 years, beginning with Norris Dam in 1933 and ending with the closure of Boone Dam December 16, 1952. As discussed in the preceding chapter, surveys of preventable damage at Chattanooga and at other cities were made at various times during this period, but the most complete survey coincident with the end of the construction periods was made in 1953. Accordingly, flood control benefits for the purposes of this report are based on the state of development as of 1953.

BENEFITS IN THE TENNESSEE VALLEY

Highest flood for benefit determination

In any discussion of the benefits from flood control by reservoirs, the question immediately arises as to the magnitude of the flood which should be considered in estimating the upper and lower limits of flooding. It appears that the occurrence of the maximum probable flood—used by TVA for the design of spillways—is too remote to be considered. In addition, this is hypothetical insofar as the distribution of rainfall and consequent runoff from the various tributaries is concerned. Also, there are of course no high water marks for this flood, so that stages in most instances must be computed for an assumed runoff greater than has ever occurred.

On the other hand, the maximum flood of record has actually occurred and is the highest one considered by TVA in the determination of benefits. On some headwater streams the maximum known discharge approaches the maximum probable flood, as in the March 1929 flood on the Emory River.

Unprotected areas

In the Tennessee River Basin there are quite a number of both large and small tributaries where no reservoirs or other facilities have been built for flood control. Consequently, there are no means of reducing floods on these streams, and hence there are no benefits except where the tributary in question enters the main river not far below a dam on the latter. In this case, regulation of the flood on the main stream will lower it at the mouth of the tributary, and this reduction will be reflected for some distance upstream on the tributary.

Protected areas

Areas which do receive protection from the reservoirs may be either substantially fully protected or only partially protected, as described in the following paragraphs.

Areas given substantial year-round protection—It seems rather obvious that determination of the protected area should be based on the maximum known flood rather than on a hypothetical flood that is expected to occur only at extremely rare intervals. Occupants of the flood plain are not ordinarily concerned by thoughts of such a flood. Also, the average annual damage allotted to such a flood would not be significant because of the long time element involved, while a reservoir to control such a flood would be very costly. However, the extent of lands flooded by this maximum probable flood would be only moderately greater than for the maximum flood of record. Accordingly, the greatest flood of record was adopted as the upper limit to which benefits should be determined. The profile of this flood as reduced by the TVA reservoir system constitutes the lower limit to which full protection extends. The difference between the two encompasses the area given substantially complete protection. The area so defined is estimated to be 110,800 acres.

Partially protected land—This term is applied to land lying between the profile of the maximum flood of record as reduced by upstream reservoirs and a flat pool from the maximum allowable summer level at the dam. A considerable portion of this land can now produce more than formerly because of reduction in frequency and depth of flooding. There are 32,900 acres in this category and, in addition, there are 120,000 acres on which it has been estimated that flooding has either been eliminated or reduced in depth. This gives a total of 152,900 acres in the partially protected category on which benefits can be estimated. Most of this land and that of the preceding paragraph lie in the upper reaches of the main Tennessee River reservoirs and on the tributaries downstream from the flood control reservoirs.

Benefits to cities

Located in both the areas defined in the two foregoing paragraphs are a number of cities receiving benefits resulting from the reduction of flood crests. These benefits, which are the average annual damages preventable by the TVA reservoir system, have been determined to be $5,556,000 (see chapter 12). They are primary tangible benefits, directly related to the affected property or to the avoidance of costs connected with the flood.

Benefits to agricultural land

Also included in both areas defined above are agricultural lands, some of which are completely protected in a flood equal to the maximum crop-season flood, and some of which have the frequency or depth of flooding reduced. The amount of this crop land is estimated to be approximately 100,000 acres, and the average annual agricultural damages preventable by the reservoir system have been determined to be $363,000 per annum (see chapter 12).

BENEFITS ON THE LOWER OHIO AND MISSISSIPPI RIVERS

One of the principal purposes of TVA, as stated in the statute under which it was created, is "to control the destructive flood waters in the Tennessee River and Mississippi River Basins." To determine the value of this control, TVA, in 1936, made a comprehensive study and report of the value of reduction of flood stages on the lower Ohio River from Paducah to the mouth at Cairo, and on the Mississippi from Cairo to the mouth of Red River.[1]

This report discusses benefits resulting from flood height reduction and determines the value of the benefits for a 2-foot reduction from Cairo to the mouth of the Arkansas River, a 1-foot reduction from there to the mouth of the Red River, and no reduction at and below the Red River. Primary tangible benefits include preventable flood damages to some 4,000,000 acres in the unprotected and backwater areas. Secondary benefits are largely the estimated increase in land values of some 6,000,000 acres already protected by the levee system, but which receive greater security from floods because of upstream reservoir control.

The 4,000,000 acres lie in the unprotected areas of Western Kentucky and Tennessee, western Mississippi and eastern Louisiana, and in the backwater areas of the St. Francis, White-Arkansas, and Yazoo Rivers. The reduction of the average annual overflow area amounts to about 386,000 acres, of which 88,000 acres were cultivated at the time of the study. The preventable damage includes damage to agricultural crops, railroads and highways, seepage damage, and savings in levee maintenance. Damages to agricultural crops, of course, vary with the date of occurrence of the flood. An analysis of the material presented in the basic study resulted in a chart (figure 129, page 224) which shows the preventable damage per foot of flood crest reduction for various crest stages and dates of occurrence. The computation of the average annual damage on this land, preventable by the TVA reservoir system, is given in chapter 12. On the basis of monetary values as of 1953, these preventable damages are estimated to amount to $2,869,000 per year. Damages in floodway lands in the Birds Point-New Madrid and Eudora floodways are not included. Moreover, benefits to timberlands resulting from flood reduction, such as the possible conversion of timberlands to cultivated lands, have not been considered.

A greater feeling of security resulting from upstream reservoir regulation would be reflected in an increase value of land, and the additional security obtained is estimated to bring an increase of $25 per acre to the 6,000,000 acres now protected, or a total increase of $150,000,000. To reduce this lump-sum increase to an average annual benefit so as to make it comparable with other average annual benefits it is multiplied by 3 percent, giving an average annual benefit of $4,500,000. The selection of the multiplier of 3 percent is believed to be conservatively low if it is considered as a return on a farm investment or a business enterprise.

1. *Value of Flood Height Reduction from TVA Reservoirs to the Alluvial Valley of the Lower Mississippi River*, Technical Monograph No. 45, October 1939. House Document No. 455, 76th Congress, 1st Session.

INCIDENTAL BENEFITS TO OTHER PROGRAMS

Incidental benefits to other water use programs are achieved as a result of storing floodwater during the operation of the reservoir system for its primary purposes—navigation, flood control, and power. Municipal and industrial water supplies have been greatly enhanced in quantity and quality, and suitable sites with adequate supplies of cooling water are made available for large modern steam generating plants. For example, the low temperature of water released from Norris Dam substantially increases the operating efficiency of TVA's Kingston Steam Plant (fig. 132). The release of previously stored floodwater during periods of low flows also contributes to the improvement of navigation on the Mississippi River and to the prevention of salt water intrusion into the Mississippi from the Gulf of Mexico.

SUMMARY OF AVERAGE ANNUAL FLOOD CONTROL BENEFITS

A summary of the flood control benefits in the various areas due to TVA's flood control system is given in table 43.

TABLE 43.—*Summary of average annual flood control benefits.*

Average annual preventable damage (1953 status):[1]	
Tennessee River Basin	$5,919,000
Lower Ohio and Mississippi Rivers	2,869,000
Total	$8,788,000
Increase in land value (average annual benefit basis):	
Lower Ohio and Mississippi Rivers	4,500,000
Total average annual flood control benefits	$13,288,000

1. From table 42, "Summary of average annual flood damages preventable by the TVA reservoir system," page 225.

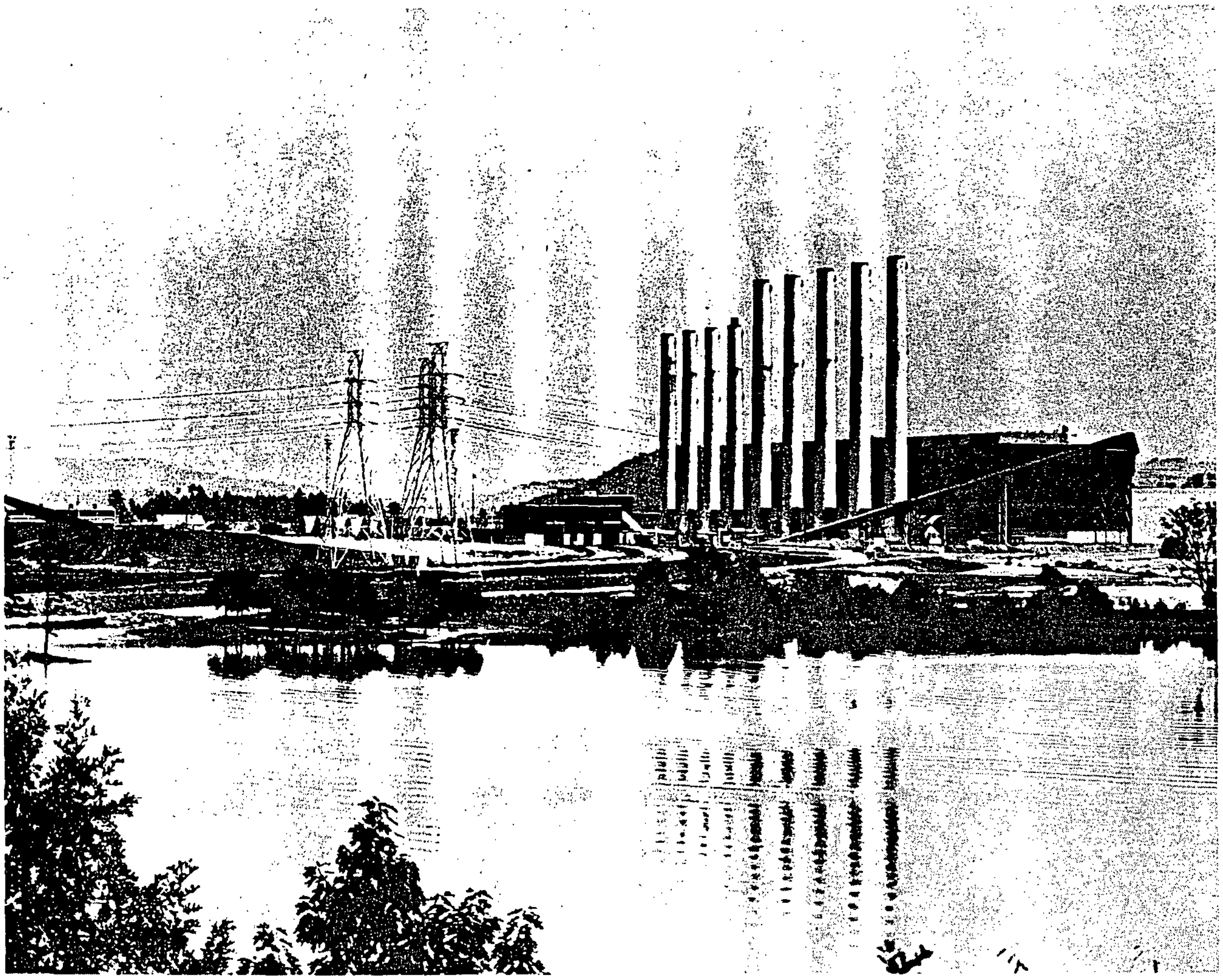


FIGURE 132.—*TVA's Kingston Steam Plant—low temperature water released from Norris Dam increases the operating efficiency of this plant.*

FIGURE 133.—*Kentucky Dam—TVA's largest multiple-purpose water control project.*

CHAPTER 14

COST AND ECONOMICS OF FLOOD CONTROL

To properly appraise the economic feasibility of a system of multiple-purpose projects, it is necessary to determine the portion of the joint costs that should be charged to the individual purposes for which the projects were authorized. In the TVA system these purposes are Navigation, Flood Control, and Power.

The TVA method of allocating joint costs is outlined in this chapter which also covers many other ramifications of flood control cost and economics. In addition to the allocation method, the chapter discussions include average annual costs, allocation of investment to flood control, annual charges for flood control, benefit-cost ratio, and return on investment. Summaries of flood control accomplishments in relation to benefits and costs, and of flood control operations as they affect water power and land, conclude this final chapter of the report.

AVERAGE ANNUAL COSTS

The cost of a proposed project should not be difficult to determine provided the requisite physical data have been obtained at the site for preparation of an adequate plan for construction. On the other hand, conversion of capital cost to an average annual cost basis for use in the benefit-cost analysis requires a great deal of judgment. It involves fixing an interest rate, selecting the length of the useful economic life of the project as a basis for amortization of the cost; estimating long-time operation costs; and estimating costs of maintaining the project at full operating capacity during its economic life. In its planning studies and reports, TVA has generally used an average interest rate of 2.5 percent and a useful economic life of 50 years. Operation and maintenance costs are based on those projects that have been constructed and are already in operation.

TVA METHOD OF ALLOCATION OF COST OF MULTIPLE-PURPOSE PROJECTS

Section 14 of the TVA Act, as amended by 48 Stat. 66, provides that:

The Board shall make a thorough investigation as to the present value of Dam Numbered 2, and the steam plants at nitrate plant numbered 1, and nitrate plant numbered 2, and as to the cost of Cove Creek Dam, for the purpose of ascertaining how much of the value or the cost of said properties shall be allocated and charged up to (1) flood control, (2) navigation, (3) fertilizer, (4) national defense, and (5) the development of power. The findings thus made by the board, when approved by the President of the United States, shall be final, and such findings shall thereafter be used in all allocation of value for the purpose of keeping the book value of said properties. In like manner, the cost and book value of any dams, steam plants, or other similar improvements hereafter constructed and turned over to said board for the purpose of control and management shall be ascertained and allocated.

TVA's latest allocation report was made as of June 30, 1953, and submitted to the President December 15, 1953. It was approved by him January 21, 1955, and is included herein as Appendix D.

In accordance with the above provision of the TVA Act, a method of allocation of the joint costs known as the "Alternative Justifiable Expenditure" method was devised by TVA after a great deal of study and investigation.

According to this method, the direct cost for any one purpose corresponds to the investment that could have been eliminated from the total project cost if that purpose had not been included in the project. The "alternative justifiable" expenditure for any one purpose is the lowest cost of realizing an equal benefit to that obtained in the multiple-purpose project by a development undertaken solely for that single purpose, provided such expenditure is justified by the benefits obtainable. The remainder, obtained by deducting the direct cost from the alternative justifiable cost for that purpose, is the maximum amount which could be justifiably contributed by that purpose toward the joint cost of the multiple-purpose project. The total common or joint cost is then divided in proportion to these remainders.

Any method of allocation is admittedly an approximation. It should not be assumed as supplying a precise answer. Accordingly, TVA rounds out the final percentage charged to each purpose. For this reason and others, TVA allocates multiple-purpose projects on a system basis and not by individual projects. These reasons are as follows:

1. There are no multiple-purpose projects in the system which do not share the benefits they produce with other projects, both existing and future. It is obviously impossible to segregate these benefits, by projects, with any degree of accuracy.
2. The amount that is allocated to any one purpose in a given project cannot be obtained by taking the incremental difference in allocations before and after a project is added, for the reason that this step involves taking small differences between large sums which are not mathematically precise.
3. Since the allocation figures as calculated are largely a guide to judgment, the final allocation actually used for the system may vary considerably from the theoretical calculations.

The first allocation for the multiple-purpose system, consisting of the Wilson, Wheeler, and Norris water control projects, was made in 1937 and approved by the President June 16, 1938. Thereafter, allocations were made as of June 30, at the end of the fiscal year following the commencement of operation of one or more multiple-purpose projects. The latest allocation was made as of June 30, 1953 (see Appendix D). At that time there were 19 multiple-purpose projects in operation, the last of which, Boone, was placed in operation during that fiscal year, and one of which, Hales Bar, has no flood storage reservation. In these allocations, no costs were charged to fertilizer or national defense. The fertilizer plant has no features in common with navigation, flood control, or power, and pays for any power it uses. Likewise, any national defense activities at the fertilizer plant near Wilson Dam were supported by the national defense budget.

ALLOCATION OF INVESTMENT TO FLOOD CONTROL

The direct flood control investment at any project is the cost of facilities specifically provided for the purpose, such as sluiceways, and includes the cost of increased height of dam and reservoir facilities necessary to provide storage space in addition to that normally required for the other purposes. Such cost for flood control may be determined by deducting the estimated cost of a hypothetical dual-purpose project, designed for navigation and power, from the cost of the multiple-purpose project as constructed.

At each main-river project, the height of a dual-purpose structure at that site is fixed by the normal maximum operating level for navigation and power under multiple-purpose operating schedules. At each storage project located on the tributaries, the height

TABLE 44.—*Reservoir elevations.*

Project	Elevation top of gates	Flood season: Normal multiple-purpose operating levels		After the flood season: Normal maximum operating levels, main-river dams	After the flood season: Average maximum elevation reached annually, tributary dams
Main river:					
Kentucky	375	354		359	
Pickwick Landing	418	408		414	
Wilson	507.88	504.5		507.5	
Wheeler	556.3	550		556	
Guntersville	595.44	593		595	
Hales Bar	635	632		634	
Chickamauga	685.44	675		682.5	
Watts Bar	745	735		741	
Fort Loudoun	815	807		813	
Tributary:		On January 1	On March 15		
Norris	1034	978	990		1005
Cherokee	1075	1020	1042		1061
Douglas	1002	935	958		986
Fontana	1710	1615	1644		1682
Hiwassee	1526.5	1455	1472		1515
Chatuge	1928	1910	1916		1922
Nottely	1780	1743	1755		1770
Watauga	1975[1]	1934	1952		1950
South Holston	1742[1]	1702	1713		1720
Boone	1385	1358	1375		1385

1. Spillway crest.

of a dual-purpose structure at that site is determined by the average of the maximum elevations to which the multiple-purpose reservoir fills annually for navigation and power purposes, after having observed the limiting multiple-purpose operating guide during the flood season. The main-river reservoirs nearly always reach the planned normal maximum operating levels shown in table 44. The average maximum elevations reached in the tributary reservoirs and the limiting March 15 levels are also shown in this table.

Alternative costs are based on estimates of the most economical system of single-purpose structures which would furnish substantially the same quantity and quality of service for the single purpose as that provided for that purpose by the multiple-purpose system. As most of the alternative projects were assumed to be built at sites of actual multiple-purpose projects, the estimates generally reflect actual knowledge of construction conditions. Estimates for single-purpose projects were based on construction cost levels experienced at the time of construction of the corresponding multiple-purpose project.

The hypothetical alternative single-use system for flood control includes 3 reservoirs on the main river and 8 on the major tributaries. These 11 reservoirs would provide flood control storage equal in total amount and effectiveness to that provided by the 18 multiple-purpose reservoirs as built. The location, amount of storage provided, and the estimated cost of the alternative projects are shown in table 45. The elevation of the top of spillway gates for each of these hypothetical structures is also shown. The average cost per acre-foot for flood control storage, $22, in the alternative single-use system is obtained and applied to the actual amount of storage capacity available in the multiple-purpose system, as shown in table 46, to determine the alternative justifiable expenditure for flood control of $260,458,000.

TABLE 45.—*Estimated cost of storage in single-use flood control system.*

Location of alternative single-use structure	Flood control storage available, acre-feet	Elevation top of spillway gates	Estimated cost of project
Tributary sites:			
Norris	1,635,000	1007.5	$ 19,000,000
Hiwassee	398,000	1520	12,501,000
Fontana	771,000	1632	27,532,000
Nottely	110,000	1758.4	4,532,000
Cherokee	1,146,000	1060	21,302,000
Douglas	1,311,000	995	30,694,000
Watauga	260,000	1900[1]	15,044,000
South Holston	400,000	1691.5[1]	18,125,000
Main-river sites:			
Watts Bar	844,000	745	16,538,000
Wheeler	541,000	545	17,348,000
Kentucky	4,477,000	375	78,554,000
Total	11,893,000		$261,170,000

Average cost per acre-foot $22.00

1. Spillway crest.

TABLE 46.—*Alternative justifiable expenditure for flood control.*

Name of multiple-use project	Elevation of top of spillway gates	January 1 elevation (flood schedule)	Flood control storage available[1], acre-feet
Kentucky	375	354	4,011,000
Pickwick Landing	418	408	418,000
Wilson	507.88	504.5	53,000
Wheeler	556.3	550	349,000
Guntersville	595.44	593	163,000
Hales Bar	635	—	—
Chickamauga	685.44	675	329,000
Watts Bar	745	735	378,000
Fort Loudoun	815	807	109,000
Norris	1034	978	1,635,000
Watauga	1975[2]	1934	260,000
South Holston	1742[2]	1702	300,000
Cherokee	1075	1020	1,146,000
Douglas	1002	935	1,311,000
Fontana	1710	1615	771,000
Hiwassee	1526.5	1455	291,000
Chatuge	1928	1910	105,000
Nottely	1780	1743	110,000
Boone	1385	1358	100,000
Total			11,839,000

Alternative justifiable expenditure for flood control
11,839,000 x $22.00 = $260,458,000

1. Any differences between these storages and those shown in table 26 are due to "rounding" or to minor changes in volume curves. The quantities given in table 26 are the most recent determinations.
2. Spillway crest.

Table 47 shows the calculation of the percentages for guidance in allocation of the common costs of the multiple-purpose system as of June 30, 1953, to the three purposes—navigation, flood control, and power. These percentages were then rounded to 27, 31, and 42, respectively, and applied to the common costs of 19 multiple-purpose projects as of June 30, 1958. Table 48 shows the calculation of the total system investment as of that date in the three purposes by the addition of direct costs at multiple-purpose projects; allocated costs at multiple-purpose projects; and single-purpose projects and other electric plant.

The foregoing tables show a total investment for flood control of $184,082,520 as of June 30, 1958, and the method by which this was obtained. The application of 31.0 percent to the common cost (adopted in the 1953 allocation) results in the allocation of $127,961,554 to flood control. The direct investment of flood control works in service was $56,120,966 as of June 30, 1958, making the total system investment for flood control equal to $184,082,520, or 8.8 percent of the total system investment for all purposes as of that same date.

TABLE 47.—*Allocation of estimated multiple-purpose system costs upon completion of work in progress as of January 3, 1953.*

Purpose	Calculation of percentages for guidance in distribution of common costs				Allocation of common costs	Total estimated multiple-purpose investment
	Alternative justifiable expenditures	Direct investment	Remaining alternative justifiable expenditures			
			Amount	Percent		
Navigation	$ 231,818,000	$ 46,168,032	$185,649,968	27.6	$113,760,303	$159,928,335
Flood control	260,458,000	55,439,000	205,019,000	30.4	125,301,203	180,740,203
Power	513,822,000	230,779,799	283,042,201	42.0	173,113,505	403,893,304
Total	$1,006,098,000	$332,386,831	$673,711,169	100.0	$412,175,011	$744,561,842

TABLE 48.—*Allocation of investment in plant in service as of June 30, 1958.*

Purpose	Direct investments	Allocation of common costs		Single-purpose projects and other electric plant	Total system investment	
		Percent	Amount		Amount	Percent
Navigation	$ 46,927,296	27.0	$111,450,387	$ 931,698	$ 159,309,381	7.6
Flood control	56,120,966	31.0	127,961,554	—	184,082,520	8.8
Power	227,915,809	42.0	173,367,269	1,353,690,482	1,754,973,560	83.6
Total	$330,964,071	100.0	$412,779,210	$1,354,622,180	$2,098,365,461	100.0

ANNUAL CHARGES FOR FLOOD CONTROL

The investment of about $184,083,000 forms the basis of calculating the annual fixed charges of interest and amortization. The average interest rate paid by the U. S. Treasury on long-term bonds over the period of construction of TVA's multiple-purpose projects was about 2½ percent, so this rate was adopted. It seems only proper in setting up the justification of such projects that an allowance for amortization be made which would equal the depreciable cost of the project in a reasonable period of time. For the purposes of this report, a period of 50 years has been set in which to amortize the depreciable items. It was assumed that the cost of lands and land rights, clearing, and relocation were nondepreciable. The nondepreciable items as of June 30, 1958, totaled $79,903,000. Since these costs are not to be amortized, interest charges should be in perpetuity.

The annual operating and maintenance costs of multiple-purpose projects are based on 7 fiscal years (1952 to 1958) actual experience. These costs are apportioned among navigation, flood control, and power, either on the basis of their relationship to the primary programs or in the same percentage as used in the allocation of capital costs.

As shown in table 49, the annual fixed charges against flood control are $5,671,000, and the average annual operation and maintenance charges are $1,101,000. The total average annual charges on this basis to flood control are $6,772,000 on a total investment of $184,083,000.

TABLE 49.—*Annual charges against flood control.*

Fixed charges:		
Interest only: $79,903,000 x 0.025		$1,998,000
Interest and amortization: $104,180,000 x 0.0352581 [1]		3,673,000
Total annual fixed charges		$5,671,000
Operation and maintenance:		
Operation	$972,000	
Maintenance	59,000	
Administration and general	70,000	
		1,101,000
Total annual charges		$6,772,000

1. Annual payment required to amortize $1 and interest at 2½ percent in 50 years.

BENEFIT-COST RATIO

The average annual flood control benefits, as measured only by the average annual preventable damage in the Tennessee River Basin and on the lower Ohio and Mississippi Rivers from the 19 multiple-purpose projects in the Tennessee River Basin, are $8,788,000, as stated in chapters 12 and 13. This results in a benefit-cost ratio of 1.30:1. If the increase in value of lands that will be completely protected by a 2-foot lowering of the maximum flood against the Mississippi levee is taken into account, this ratio becomes 1.96:1.

RETURN ON INVESTMENT

Another approach to the problem of economic justification is to calculate the annual rate of return on the investment with which to cover full charges. In this method, the net annual benefit is obtained by deducting from the benefit the average annual operation and maintenance charges (and other charges that are not to be included in the rate of return), and dividing by the capital cost of the project for which the net benefit and other fixed charges have been calculated. The formula for this is as follows:

$$\text{Percent Return} = \frac{\text{Annual benefit} - \text{average annual O \& M charges}}{\text{Capital Cost of Project}}$$

$$= \frac{8{,}788{,}000 - 1{,}101{,}000}{184{,}083{,}000}$$

$$= \frac{7{,}687{,}000}{184{,}083{,}000}$$

$$= 4.2\% \text{ to cover interest, amortization, etc.}$$

Taking into consideration the increased value of land now afforded partial protection behind the levees, the equation becomes

$$\frac{13{,}288{,}000 - 1{,}101{,}000}{184{,}083{,}000} = \frac{12{,}187{,}000}{184{,}083{,}000} = 6.6\% \text{ return.}$$

ACCOMPLISHMENTS OF FLOOD CONTROL IN RELATION TO BENEFITS AND COSTS

Preceding chapters describe the nature and extent of the protection from floods that have been afforded by the TVA reservoir system. This section summarizes these accomplishments since the start of flood control operations in 1936, and ends with a comparison of accumulated benefits with total costs.

There are now 770 miles of river below the multiple-purpose dams where the agricultural lands on the bottoms on both sides of the stream have the frequency of flooding reduced greatly.

The flood of February 1957 at Chattanooga would have reached a stage of 54 feet, the second highest of record, under natural conditions, but was actually reduced nearly 22 feet to the point where only minor damage occurred. Substantial but smaller reductions have been achieved at Chattanooga in 32 other floods since 1936. Damages averted in these floods, from the stage-damage relationship pertaining at the time, now (June 30, 1960) total $120,213,000.

Along the lower Ohio and Mississippi Rivers, substantial reductions in flood damages have been achieved in 12 of the 15 years since Kentucky Reservoir was completed in 1945. At Cairo, the great flood of January 1950 was reduced 1.9 feet. The flood of May 1958 was reduced 3.1 feet, and although it was a smaller flood than that of 1950, it was more significant in prevented damages because of the date of its occurrence. In other floods the stage at Cairo can be reduced up to 4 feet, depending on the contribution of the Tennessee River to the peak on the Ohio River. Damages averted since 1945 on the land not protected by the Mississippi River levee system now (June 30, 1960) total $24,651,000.

The total damages averted to June 30, 1960, by virtue of flood reductions in actual floods experienced since 1936 total nearly $145 million, or about three-fourths of the total capital investment of $184 million allocated to flood control. This damage total includes only the damages averted at Chattanooga and in the lower Ohio-Mississippi River Valleys, since it has not been practicable to accumulate damages averted at all other locations in the Tennessee River Basin.

The benefits to the Ohio-Mississippi region of increased land value, by virtue of greater security against floods, may be considered as accruing in total immediately upon completion of the protective works instead of on an annual basis. This benefit from the TVA system, estimated at $150 million plus the incomplete total of $145 million of damages averted, makes the total accumulated benefits to date equal to $295 million.

Flood control costs to June 30, 1960, including the capital investment of $184 million, an allowance for interest of $75 million on this investment, together with operation and maintenance expenditures of $22 million, total $281 million. Comparison with accumulated benefits thus shows that benefits over a period of 25 years are more than sufficient to amortize the entire flood control investment, plus all accumulated annual expenditures and an allowance for interest, and the system will continue to supply these benefits for many years to come.

FLOOD CONTROL OPERATIONS AS THEY AFFECT WATER POWER AND LAND

TVA, under the Act, must operate its dams and reservoirs "to regulate the stream flow primarily for the purpose of promoting navigation and controlling floods" and to produce the maximum water power "so far as may be consistent with such purposes."

The Tennessee Valley system of reservoirs has been controlling floods and producing power for many years. This is accomplished by fitting the operation of the dams into the annual rainfall and stream flow cycle. Major valley-wide floods, as shown by nearly a century of records, occur only between late December and early April. At the beginning of the flood season, about January 1 each year, nearly 12,000,000 acre-feet of storage space is reserved to regulate floods on the Tennessee and to aid the regulation of floods on the lower Ohio and Mississippi Rivers. After March 15, near the end of the flood season, the reservoirs are filled as rapidly as possible, although about 2,500,000 acre-feet is always reserved for flood control.

In the drier periods of late summer and fall the water thus stored is released to maintain the flow of the streams for power production and navigation. These releases automatically lower the levels of storage reservoirs to provide space for the control of next season's floods. TVA strictly observes the flood control priority. For example, more than a million acre-feet of water which had been stored to regulate a major flood in February 1957 was spilled from the reservoirs in order to return them to required flood control levels; except for the flood control priority this water could have been retained to generate power.

The land in the reservoirs is used not only for flood control, but also for navigation and power production and for such additional benefits as recreation, fish and wildlife propagation, improved water supplies, and control of malaria. Of the 606,000 acres of land inundated if and when the reservoirs are filled to their upper levels for flood control, 327,000 acres (54 percent) are devoted solely to navigation and power purposes. The land in the reservoirs used solely for flood control or jointly for flood control and power amounts to less than 279,000 acres, of which 47 percent is in Kentucky Reservoir for use in reducing Mississippi River floods.

As stated in chapter 13, the system has completely eliminated damages from floods as great as the maximum known on more than 110,000 acres of land in the Tennessee Valley, and it aids in holding floods below the tops of levees in protecting more than 6 million acres of productive land along the Mississippi River. It can also reduce the depths and frequency of floods on additional lands conservatively estimated at more than 153,000 acres in the Tennessee Basin and on 4,000,000 acres along the Mississippi.

This simple acreage comparison does not tell the entire story, however. By the planned and orderly impoundment of water in deep reservoirs, largely on low-value land, TVA protects other areas of productive farm lands and high-value urban properties from damaging floods. Cost of land purchased for reservoir purposes has averaged about $74 per acre. This includes a great deal of land not actually covered by water. In comparison, the second largest flood of record, which occurred in February 1957, would have caused damages of $7,950 an acre in the City of Chattanooga; except for the TVA system, it would have covered 8,300 acres with total damage estimated at $66 million. The flood of 1867, the maximum known, would cover 9,000 acres and cause $105 million damage, an average of $11,000 an acre. The reservoir system could reduce this to about $12 million, and it makes practicable the building of levees which would fully protect the city against a flood 60 percent greater than that of 1867. The foregoing damage estimates for Chattanooga are based on its 1953 status.

Sometimes it is claimed that the elimination of agricultural production on lands used in reservoirs cancels flood reduction benefits. Much of the area in reservoirs was low-value hillside land and cutover timberland. Actually, the land is more productive now than it was in agricultural use. The multiple-purpose water control system of which the land is an essential part produces about $150 million worth of electric power (what the consumers pay for it) in an average year, savings in freight charges due to navigation verge on $25 million a year, and average annual benefits from flood reduction are over $13 million, without counting the several other benefits of river regulation.

ACKNOWLEDGMENTS

This report is a product of the efforts and experience of the Tennessee Valley Authority in carrying out its flood control program. It was compiled and edited by James S. Bowman, former Chief Water Control Planning Engineer, who also wrote major portions of the text. Chapters 2, 3, 11, and parts of chapters 4 and 5, all relating to flood storms, flood histories, or effect of changes in land use, were written by Albert S. Fry and James Smallshaw of the Hydraulic Data Branch. The remaining portions of chapters 4 and 5, and chapters 6, 8, 10, 12, appendix B, and portions of chapters 7, 13, and 14, all relating to Valley-wide and local flood problems, were written by Edward J. Rutter and Bob J. Buehler of the Flood Control Branch. Major portions of chapter 7 and all of chapter 9 and appendix A, which relate to operations for flood control, were written by Nicholls W. Bowden, Le Roy Engstrom, and Alfred J. Cooper of the River Control Branch.

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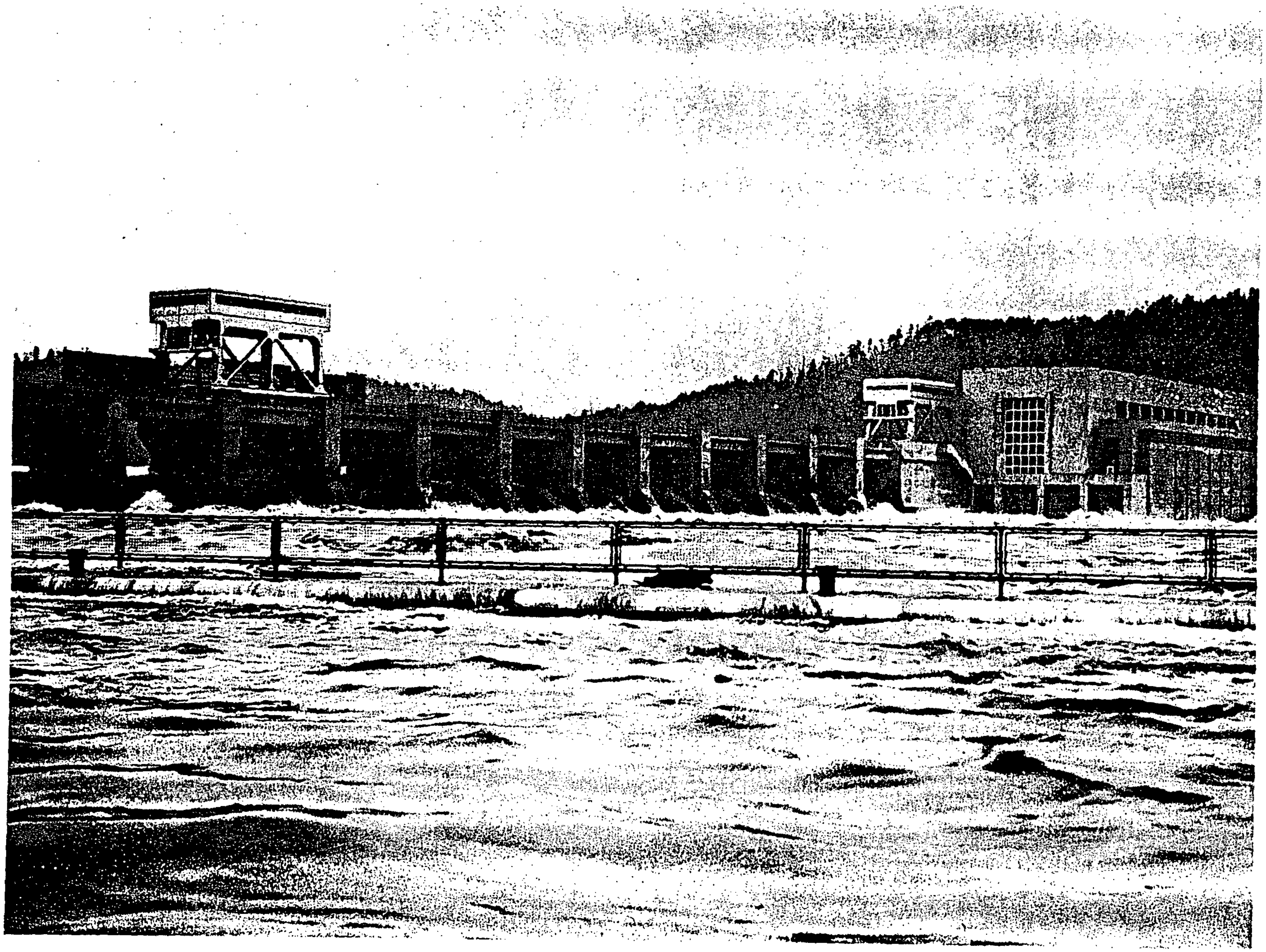


FIGURE 134.—*Guntersville Dam releasing 250,000 cfs during 1946 flood control operation.*

APPENDIX A

TYPICAL FLOOD CONTROL OPERATION JANUARY 1947 AND RESULTS OF FLOOD REGULATION JANUARY 1946

This appendix is devoted primarily to the day by day regulation of the flood produced in the Tennessee Basin by the storm of January 14-20, 1947. This regulation is a typical example of TVA's flood control operations and of their coordination with U. S. Weather Bureau forecasts. The benefits effected by this 1947 flood regulation at such key points as Chattanooga, Paducah, and Cairo, as well as reduction of flooding elsewhere, are also indicated. Because these operations during the 1947 flood were in close comformity with the principles enumerated in chapter 7, they should be considered in the light of those principles and also in the light of their being by necessity "foresight" and not "hindsight" operations.

The successful regulation of the even higher 1946 flood was also a noteworthy operation and a brief summary of this operation and the results achieved appear at the end of this appendix.

The emphasis of the 1947 flood regulation was to reduce the crest at Chattanooga as much as possible. Therefore, before proceeding with the description of flood control operations for the 1947 flood it is appropriate to present the following discussion concerning the selection of those points or locations where maximum flood regulation is to be effected along the flood carrying stream.

Selection of points for maximum flood regulation

The selection of points for effecting the maximum regulation depends upon the location and magnitude of the flood to be regulated. At times, particularly in summer, local headwater floods are more severe because of rain such as in the West Indian hurricanes over relatively small areas. One or more of the upper tributaries may become flooded by such a rain and regulatory operations reduce the stages below affected tributary dams with the regulatory effect decreasing progressively downstream. Even without regulation, Chattanooga will not usually reach flood stage from this type of flood.

At other times, only the lower mainstream below Chattanooga is affected materially, the storms being over the western half of the Basin. Regulation is then directed to effect the maximum benefits for points including farmlands in and around Decatur and Florence, Alabama, and Savannah, Tennessee.

When a general flood occurs covering all or most of the Basin, regulation is then directed toward reduction of flows on the entire system, with emphasis on areas subject to major flood damages, as Chattanooga and Cairo. Such reductions will also usually produce reductions above and below those points. The January 1947 flood is an example of this general type of flood.

FLOOD CONTROL OPERATIONS JANUARY 1947

Rainfall, runoff and reservoir conditions—January 1-14

Intermittent rainfall during the first 14 days of January 1947 produced high groundwater levels and developed conditions favorable to a high runoff. However, on January 14 all tributary reservoirs were well below maximum multiple-purpose levels except Douglas which was almost 3 feet above this level. The mainstream reservoirs were within the usual winter fluctuation range with the exception of Pickwick Landing Reservoir. This reservoir was in the process of being returned to the range after use in regulation of flows during the first half of January. At this time, the stage was 23.8 feet at Paducah on the Ohio River and 29.3 feet at Cairo on the Mississippi River. Both were well under flood stage and had a falling tendency.

Preparation for possible flood control operations

January 14 (Tuesday)—The Preliminary Forecast issued by the Knoxville office of the U. S.

Weather Bureau on Tuesday morning, January 14, predicted:

. . . Precipitation amounts during this (forecast) period (36 hours from 6:00 a.m. Tuesday to 6:00 p.m. Wednesday) will average from 0.20 to 0.40 of an inch over the western and central thirds (of the basin) and from 0.50 to 0.75 of an inch over the eastern third (of the basin) with local high spots ranging up to 1.50 inches over the Southeastern Section.

On the basis of this forecast and estimates of runoff from observed rainfall to 6:00 a.m., discharges from mainstream reservoirs were set to continue drawing them slightly in preparation for possible flood control operations, and the turbine use necessary for power production was continued at tributary reservoirs. No substantial changes in amounts of predicted rainfall were made in the Regular Forecast received shortly before noon nor in the Supplementary Forecast received that evening. Therefore, the scheduled operations set in the morning were maintained.

Mainstream drawdown in advance of flood

January 15 (Wednesday)—Precipitation observed for the 24-hour period ending at 6:00 a.m January 15 averaged 1.1 inches above Chattanooga with high spots up to 2.5 inches, and 0.5 inch below Chattanooga with high spots up to 3.0 inches. In general, it was raining throughout the basin at observation time, and the 24-hour fall had exceeded the 36-hour forecast.

The Preliminary Forecast received early Wednesday morning predicted:

. . . Precipitation will average from 0.50 to 0.75 of an inch over the western half and 0.75 to 1.00 inch over the eastern half with amounts in showers and thunderstorms ranging up to 2.50 inches, mostly over the Southwestern and Southeastern Sections.

Initial increases in discharges from mainstream reservoirs were made at 9:00 a.m. Preliminary estimates of runoff indicated such increases would be necessary if no time was to be lost in getting the

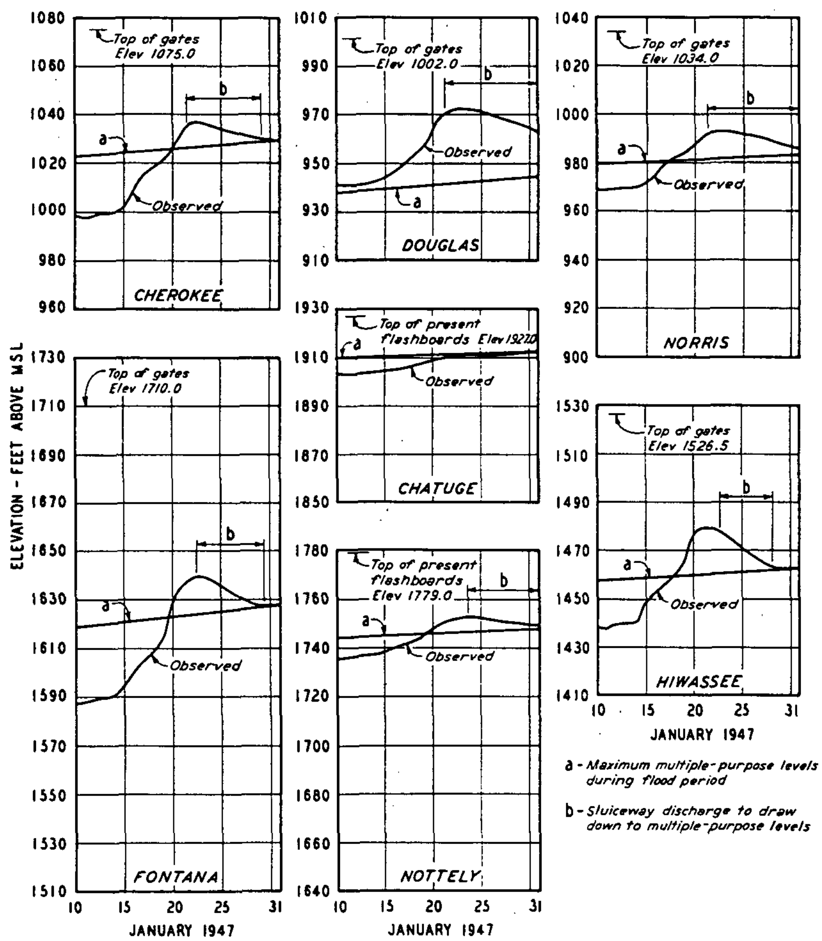


FIGURE 135.—*Tributary reservoir elevations—flood of January 1947.*

reservoir system ready to control a large winter flood, an ever-present possibility. Guntersville discharge was initially increased from 50,000 to 70,000 cfs, Wheeler from 55,000 to 75,000 cfs, and Pickwick from 82,000 to 100,000 cfs. Tributary reservoirs were permitted to store the increasing inflows in excess of turbine requirements with a resulting rise in headwater elevations as indicated in figure 135.

The first mainstream reservoir routing computations, shown at the left in figure 110, page 176, were based upon carefully considered estimates of runoff from the rainfall observed to 6:00 a.m. January 15. They indicated that a further increase in discharge would be necessary from all mainstream reservoirs to effect drawdown from the elevation shown in figure 136.

The Regular Forecast received just before noon predicted:

> . . . Precipitation amounts . . . will average from 1.25 to 1.50 inches over the East Central Section and western portion of the Southeastern Section, from 0.75 to 1.00 inch elsewhere over the eastern half, from 0.50 to 0.75 of an inch over the West Central and Southwestern Sections, and less than 0.50 of an inch over the Western Section, with local high spots up to 3.00 inches in thunderstorm areas.

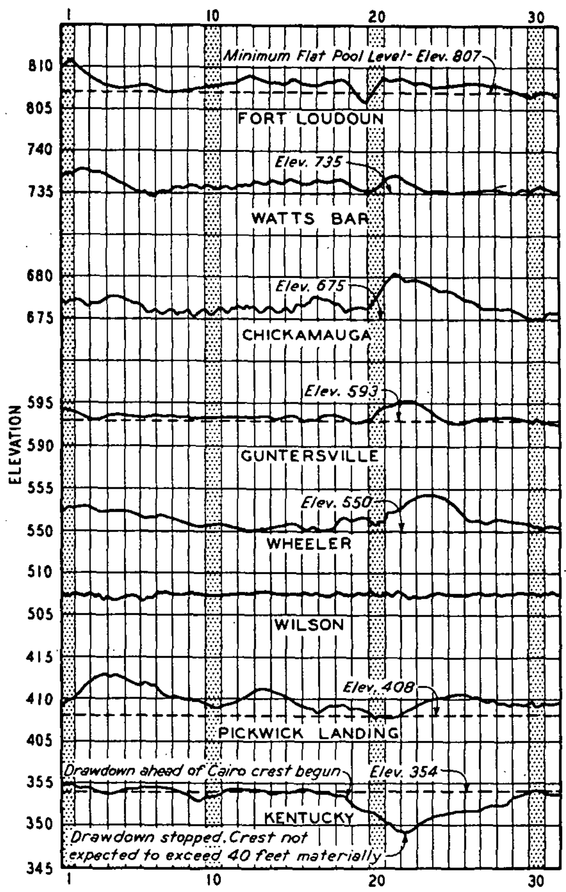


FIGURE 136.—*Mainstream reservoir elevations—flood of January 1947.*

This forecast also gave the following outlook:

> Present indications are favorable to the development of another wave disturbance which . . . (will) produce additional precipitation Thursday night. No definite indication of fair weather within the next five days. . . .

Accordingly, about noon January 15 instructions were given to the Division of Power System Operations to increase Fort Loudoun discharge from 15,000 to 30,000 cfs, Watts Bar to spill 30,000 cfs above turbine requirements, Chickamauga to increase from 39,000 to 100,000 cfs, Guntersville from 70,000 to 110,000 cfs, Wheeler from 75,000 to 110,000 cfs, Pickwick from 100,000 to 120,000 cfs, and Kentucky from 77,000 to 110,000 cfs.

Computations of natural flows indicated a daily average for January 14 of 43,000 cfs at Chickamauga Dam and 119,000 cfs at Kentucky Dam. The average system inflow for that day for the total drainage area above Chickamauga Dam was 51,000 cfs and above Kentucky Dam, 92,000 cfs.

Arrangements were made to report the additional rainfall which had fallen between 6:00 a.m. and 12:00 noon in order to check the weather forecasts and make any necessary adjustments in discharges. Routing computations were made that afternoon to determine what elevations would be reached in the mainstream reservoirs if the predicted rainfall should develop and if the discharges set at noon were maintained (fig. 110, page 176).

A conference of the engineers in charge of water control was held to review the results of the afternoon computations. It was decided to increase discharges to draw Fort Loudoun, Watts Bar, and Chickamauga Reservoirs about 1 foot below the elevations indicated for midnight January 16 in the afternoon computations and, at the same time, to regulate the flows above Chattanooga, and to draw Guntersville, Wheeler, Pickwick, and Kentucky to about minimum multiple-purpose pool levels. A revised mainstream reservoir routing computation was made to conform to these several requirements as indicated on the right in figure 110.

Based upon these revised computations, operators were instructed to begin increasing discharges at mainstream dams immediately and to maintain turbine requirements at tributary plants, storing excess water in those reservoirs. Fort Loudoun was increased to 35,000 cfs; Watts Bar to 90,000 cfs; Chickamauga, Guntersville, and Wheeler to 125,000 cfs; and Pickwick to 135,000 cfs. Kentucky was maintained at 110,000 cfs. In making all increases or decreases in discharge at the various dams a limit was fixed upon the allowable increment per hour to avoid too rapid a change in stages downstream. Predictions of stages at important stations on the Tennessee River, based upon operations and currently anticipated runoff, were issued to the U. S. Weather Bureau for distribution to the public.

The evening checkup of weather developments indicated that about 0.6 of an inch had fallen since 6:00 a.m. that morning with high spots of 1.3 inches above Chattanooga, and 0.6 of an inch with high spots of 2.3 inches below Chattanooga. It was not raining at the 6:00 p.m. observation time except in the extreme Northeastern Section.

The Supplementary Forecasts received that evening predicted:

> . . . Precipitation amounts for the next 24 hours (from 6:00 p.m. Wednesday) will average 0.50 over the western third, under 0.50 of an inch in the central third, and of little consequence in the eastern third.

No adjustments were made in discharges set earlier.

January 16 (Thursday)—Precipitation observed for the 24-hour period ending at 6:00 a.m., Thursday, January 16, averaged about 0.6 inch with high spots of 1.6 inches above Chattanooga, and averages about 0.7 inch below Chattanooga with high spots of 2.3 inches. Rain continued at observation time in all except the eastern third of the basin.

The Preliminary Forecast received early Thursday morning, January 16, predicted:

> . . . Precipitation will average from 0.40 to 0.60 of an inch over the western and central thirds, and . . . from 0.50 to 0.70 of an inch over the eastern third with a few high spots in heavy showers ranging up to 2.00 inches.

At 10:19 a.m. the U. S. Weather Bureau telephoned the river control offices and increased materially these preliminary estimates of anticipated rainfall amounts. The Regular Forecast received before noon Thursday confirmed the earlier telephone forecast:

> . . . Precipitation amounts . . . to average from 1.25 to 1.50 inches over the central portion, 1.00 to to 1.25 inches over the western portion, . . . (and) Southeastern Section, and from 0.50 to 1.00 inch over the Eastern and Northeastern Sections.

A possibility was indicated in the outlook that another period of precipitation confined mostly to the eastern half might occur Saturday, due to the chance of another wave disturbance forming over southern Texas and moving northeastward, but it appeared then unlikely to affect the Tennessee Valley area. Scheduling of mainstream reservoir releases and elevations by routing computations was made based upon runoff from an amount of rainfall equal to the sum of the rainfall observed to 6:00 a.m. and 1.00 inch additional anticipated rainfall. Tributary reservoirs were permitted to continue filling while discharging the relatively small amount of water necessary for power production (fig. 135).

Incomplete development of rainfall anticipated on Wednesday resulted in correspondingly less runoff in Fort Loudoun, Watts Bar, and Chickamauga Reservoirs. It was decided to draw Chickamauga Reservoir to elevation 675 by Saturday, January 18, in preparation for the reduction of the crest stage at Chattanooga and to hold other reservoirs at their current elevations (6:00 a.m., January 16) which are shown in figure 136.

Routing computations revealed that Fort Loudoun discharge could be reduced at noon Thursday, January 16, from 35,000 to 26,000 cfs; Watts Bar from 90,000 to 65,000 cfs; and Chickamauga from 125,000 to 115,000 cfs; and still reach these desired levels. Guntersville was increased from 125,000 to 150,000 cfs and Wheeler from 125,000 to 180,000 cfs. Computations indicated that Pickwick Landing Reservoir could be increased to a discharge which would hold it near the current elevation (at 6:00 a.m., January 16). To do so, however, would require such a high discharge that subsequently an exceedingly large reduction would be necessary to prevent drawing the reservoir below that elevation. Such reduction would give excessive fluctuations at points downstream, an undesirable feature from the standpoint of navigation and bank erosion. To prevent this the discharge of 135,000 cfs was increased to only 180,000 cfs which would result in temporarily storing some of the inflow, but from which rate, with currently anticipated runoff, desirable reductions could be made to pass the inflow in maintaining near minimum multiple-purpose pool level. The discharge from Kentucky Reservoir was increased from 110,000 to 150,000 cfs to prevent the reservoir elevation from rising above a level of 354.0 at the dam.

Computations of natural flows indicated a daily average for January 15 of 59,000 cfs at Chickamauga Dam and 108,000 cfs at Kentucky Dam. The average system inflow for that day above Chickamauga Dam was 164,000 cfs and above Kentucky Dam 277,000 cfs.

About 4:20 p.m. the U. S. Weather Bureau, in conference telephone conservation, indicated that a check had been made on the position of the cold front, and the formation of the wave disturbance mentioned in the Regular Forecast was definite. It was expected to follow a path extending eastward about 50 miles from a line between Pickwick Landing Dam northeastward to Nashville. The rainfall in this area was expected to be double the earlier predicted amounts with the greater portion falling during the day Friday, January 17. Amounts of rainfall above Chattanooga were expected to be relatively light. It was indicated that more definite information as to amounts would be available by morning and should the wave move slowly, heavy amounts would fall above the tributary dams. In order to reduce the volume of water to be regulated in Guntersville Reservoir and below, Fort Loudoun was reduced from 26,000 to 21,000 cfs, Watts Bar from 65,000 to 55,000 cfs, and Chickamauga from 115,000 to 95,000 cfs. Discharges set previously at downstream dams were continued.

The evening check-up indicated that an average of about 0.6 of an inch with high spots of 1.2 inches had fallen since 6:00 a.m. that morning above Chattanooga, and 0.5 of an inch with high spots of 0.9 of an inch below Chattanooga. It was raining over the southern half of the Basin at the time of observation. The Supplementary Forecast received Thursday night predicted:

> . . . rain ending Friday evening . . . precipitation amounts during the next 24 hours will range from 1.50 inches along the southern edge of the Basin to near 0.75 in the northern limits. An average of 1.50 inches . . . in the west and central thirds and . . . 1.00 inch in the eastern third with high spots in Southeastern Section and southern portion of West Central Section up to 2.00 inches.

No adjustments in discharges from mainstream reservoirs were necessary that night.

January 17 (Friday)—Rainfall for the 24-hour period ending at 6:00 a.m. Friday, January 17, averaged about 0.6 inch both above and below Chattanooga, with high spots of 1.4 inches above and 1.1 inches below Chattanooga. Rain was falling at observation time only in the East Central, Southeastern, and Eastern Sections.

The Preliminary Forecast early Friday morning, January 17, predicted:

> . . . Rain will end during the night, except over the extreme southeastern limits. . . . Saturday . . . scattered showers over the extreme southeastern limits, ending during the day. . . . Precipitation amounts will average from 0.30 to 0.50 of an inch over the eastern half and from 0.20 to 0.40 of an inch over the southern half of the western half. A few local high spots ranging up to 1.50 inches over the eastern half.

The first reservoir routing computations for the scheduling of releases were based upon the inflows estimated from just the observed rainfall because only light showery precipitation was predicted. The objective was to hold some mainstream reservoirs at bottom while drawing the others to minimum flat pool levels by midnight Saturday, January 18. Tributary reservoirs continued to store water while meeting power requirements. Fort Loudoun discharge was maintained at its existing 23,000 cfs; Watts Bar was increased to 64,000 cfs; Chickamauga to 115,000 cfs; Guntersville to 165,000 cfs; Wheeler was continued at 180,000 cfs; Pickwick was increased to 200,000 cfs; and Kentucky to 180,000 cfs.

With a large volume of water already moving through the reservoirs this schedule of releases would flood some of the uninhabited agricultural lowlands below each of the dams involved, but to heights materially lower than would have occurred under natural conditions without reservoir control. To enable farmers and others to remove livestock, machinery, or other property from these lowlands, the Weather Bureau broadcast advisory warnings to the people living in the vicinity of these areas, using more than twenty-one radio stations scattered throughout the Valley.

About noon Friday, January 17, the U. S. Weather Bureau forecaster said that the existing weather situation bore a striking analogy to the great storm of March 27-30, 1886, which had produced the third highest flood of record at Chattanooga. The synoptic maps of the March 1886 storm and the January 1947 storm were similar as may be seen in figure 137.

The Regular Forecast which had just been issued predicted:

> . . . Precipitation will average from 0.50 to 0.75 of an inch over the Western Section, 0.75 to 1.00 inch over the West Central, East Central, and Northeastern Sections, and from 1.00 to 1.25 inches over the Southwestern, Southeastern, and Eastern Sections, and the southeastern portion of the East Central Section . . . local high spots ranging up to

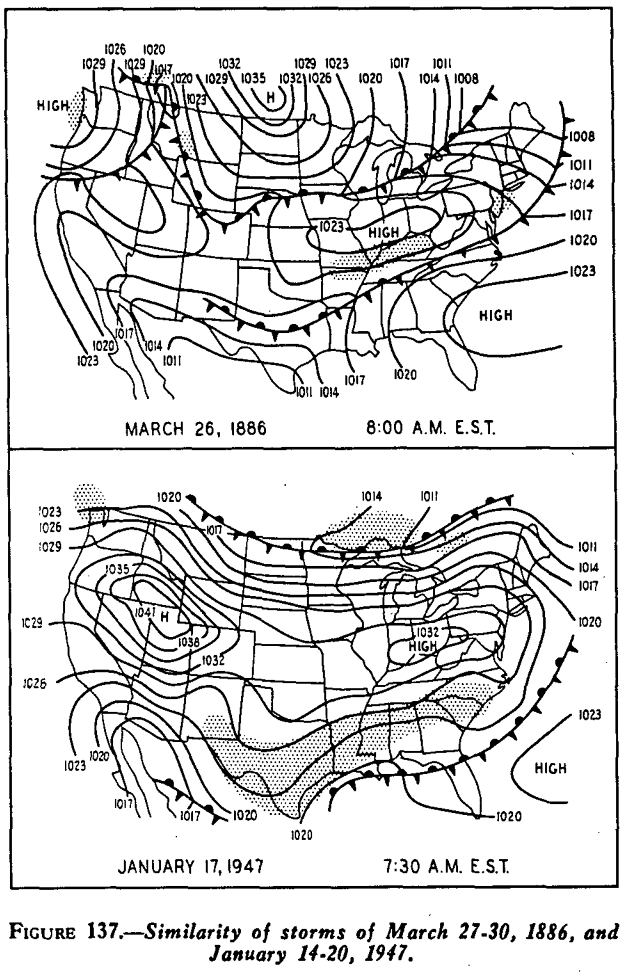


FIGURE 137.—*Similarity of storms of March 27-30, 1886, and January 14-20, 1947.*

RESERVOIR ROUTING COMPUTATIONS – VOLUMES IN 1000 DAY-SECOND-FEET

TVA 4049 (WCP-7-47) | obs. Data | Prediction based on observed rainfall to 6:00 a.m. Jan. 18 + 1.00" additional | Prediction based on assumptions to left + 1.00" additional rain beginning at 6:00 a.m. Jan. 19

	Day / Date Jan. 1947	F 17	S 18	S 19	M 20	T 21	S 19	M 20	T 21	W 22
	Cherokee	2.9	2	1	2	2	1	2	2	2
	Douglas	6.5	6	7	7	7	7	7	7	7
FORT LOUDOUN	Local Inflow	13.8	27	27	14	8	30	27	13	7
	Total Inflow	24.3	37	36	22	17	39	35	22	16
	△Storage	+2.6	+5	−4	−8	−1	−1	−5	−7	0
	Storage (Level)	146.3	151.3	147.3	139.3	138.7	150.3	145.3	138.7	138.7
	Elevation 12pm	808.26	809.1	808.4	807.1	807.0	808.9	808.1	807.0	807.0
	Outflow	21.7	32	40	30	18	40	40	29	16
WATTS BAR	Local Inflow	33.1	55	59	35	21	61	59	33	20
	Calderwood - 1 day ~~Little Tenn.~~	4.7	6	5	3	5	5	3	5	5
	Norris - 1 day	41	4	6	3	6	6	3	6	6
	Total Inflow	63.6	97	110	71	50	112	105	83	47
	△Storage	+2	+10	−10	−19	0	−8	−15	−6	0
	Storage (Profile)	408	412	402	383	383	404	389	383	383
	Elevation 12pm	736.51	737.0	736.3	735.3	735.4	736.4	735.7	735.4	735.4
	Outflow	61.6	87	120	90	50	120	120	89	47
CHICKAMAUGA	Local Inflow	24.7	44	34	21	14	40	32	19	13
	Apalachia - 1 day	2.0	2	2	0	2	2	0	2	2
	Ocoee - 1 day	6.0	6	6	7	6	7	10	9	8
	Total Inflow	94.3	139	162	118	72	169	162	119	70
	△Storage	−10	+1	+12	−12	−26	+19	+12	−11	−23
	Storage (Profile)	203	204	216	204	178	223	235	224	201
	Elevation 12pm	676.87	675.5	676.2	677.0	675.0	676.8	678.0	678.6	677.0
	Outflow	104.3	138	150	130	98	150	150	130	93
HALES BAR	Local Inflow	19.6	26	23	14	9	25	22	13	9
	compensation / Total Inflow	123.9	−6 158	173	+3 147	+3 110	175	172	143	102
	△Storage	−0.3	+6	+1	−5	−7	+2	−1	−5	−7
	Storage (Level)	83.9	90	91	86	79	92	91	86	79
	Elevation 12pm	635.9	637.5	638.0	636.5	634.5	638.2	638.0	636.5	634.5
	Outflow	124.2	152	172	152	117	173	173	148	109
GUNTERSVILLE	Local Inflow	25.9	53	37	21	14	53	40	21	13
	Total Inflow	150.1	205	209	173	131	226	213	169	122
	△Storage	−10	+17	+9	−7	−29	+26	+13	−31	−48
	Storage (Profile)	163	180	189	182	153	206	219	188	140
	Elevation 12pm	593.53	592.7	592.3	592.3	592.7	592.4	592.9	593.1	592.7
	Outflow	160.1	188	200	180	160	200	200	200	170
WHEELER	Local Inflow	48.2	61	39	27	16	65	46	24	15
	compensation / Elk	12.8	−10 18	27	+10 24	13	23	34	21	13
	Total Inflow	221.1	257	266	241	189	288	280	245	198
	△Storage	+42	+47	+26	+1	−51	+48	+40	+5	−42
	Storage (Profile)	512	559	585	586	535	607	647	652	610
	Elevation 12pm	550.70	551.6	551.5	551.5	551.8	551.7	553.4	554.6	554.0
	Outflow	179.1	210	240	240	240	240	240	240	240
WILSON	Local Inflow	2.8	13	17	9	5	18	18	9	5
	Total Inflow	181.9	223	257	249	245	258	258	249	245
	△Storage	+1.4	0	0	0	0	0	0	0	0
	Storage (Level)	35.0	35	35	35	35	35	35	35	35
	Elevation 12pm	507.55	507.5	507.5	507.5	507.5	507.5	507.5	507.5	507.5
	Outflow	180.5	223	257	249	245	258	258	249	245
PICKWICK	Local Inflow	16.2	33	29	16	12	37	31	17	12
	Total Inflow	196.7	256	286	265	257	295	289	266	257
	△Storage	+8	+31	+26	+5	−3	+35	+29	+6	−3
	Storage (Profile)	380	411	437	442	439	446	475	481	478
	Elevation 12pm	409.12	410.0	411.2	411.5	411.5	411.8	413.3	413.6	413.5
	Outflow	181.7	225	260	260	260	260	260	260	260
KENTUCKY	Local		45	55	45	30	60	70	40	25
	Back									
	Total Inflow		270	315	305	290	320	330	300	285
	△Storage		+40	+55	+45	+30	No computations			
	Storage	1225	By separate routing computations				necessary - storage			
	Elevation 12pm	353.63	352.8	351.8	351.0	351.0	space available			
	Outflow	168.8	230	260	260	260	260	260	260	260
	Tailwater		323.5	324.5	326.0					
	~~[illegible] Inflow~~									

FIGURE 138.—*January 18, 1947—mainstream reservoir routing computations.*

> 2.50 inches over the southern limits of the area occurring mostly over the Southeastern Section.

The outlook for Sunday, Monday, and Tuesday gave indications of occasional periods of moderate precipitation.

Computations of natural flows indicated a daily average for January 16 of 104,000 cfs. at Chickamauga Dam and 102,000 cfs at Kentucky Dam. The average system inflow for that day above Chickamauga Dam was 241,000 cfs and above Kentucky Dam, 424,000 cfs.

A second reservoir routing computation was made in the afternoon using estimated inflows based upon the rainfall which had fallen to 6:00 a.m., January 17, plus an additional inch of predicted rainfall assumed to fall over the entire Basin starting at 6:00 p.m. This lapse of time between observed and anticipated rainfall was to allow for the intermittent characteristics of the rainfall. This computation indicated that the discharges set about noon could be continued without further increase, except at Kentucky. There, it was necessary to increase from 180,000 to 200,000 cfs at 5:00 p.m. to prevent Kentucky Reservoir from filling above elevation 354. This was advisable in view of the rising Ohio River. Again, revised advisory warnings were issued through the Weather Bureau.

The evening check-up indicated that about 0.3 of an inch had fallen since 6:00 a.m. above Chattanooga with high spots of 0.5 of an inch, and 0.4 of an inch below Chattanooga with high spots of 1.0 inch. It was raining over all areas except over the extreme northern portion of the Western Section. The Supplementary Forecast received late Friday evening, January 17, predicted:

> . . . Precipitation amounts will average 0.50 to 0.75 of an inch in the Western Section, 1.00 to 1.50 in the Southwestern, Southeastern, and southern portion of the East Central Section. 1.00 to 1.35 inches may be expected elsewhere. High spots up to 2.50 inches . . . from Huntsville to Ocoee.

As these predicted amounts were not materially different from those amounts in the Regular Forecast and as no significant runoff-producing rain had fallen to 6:00 p.m., no adjustments in releases seemed advisable.

January 18 (Saturday)—Rainfall observed for the 24-hour period ending at 6:00 a.m., January 18, averaged about 0.9 inch above and 0.7 inch below Chattanooga. High spots of 1.7 inches were observed above Chattanooga and 1.5 inches below. Rain was occurring at observation time throughout the Basin except in the northwestern corner.

The Preliminary Forecast received early Saturday morning, January 18, predicted:

> Precipitation amounts . . . to average . . . 1.20 to 1.50 inches over the Southwestern and Western Sections, western half of the Southeastern Section, and southern half of the East Central Section, 0.9 to 1.20 inches over the West Central and Eastern Sections, northern half of the East Central Section, and the eastern half of the Southeastern Section, and 0.6 to 0.9 of an inch over the Northeastern Section . . . additional precipitation Sunday night and Monday.

Because of the observed rainfall and predicted heavy rainfall, the discharges from mainstream reservoirs were increased before any preliminary routing computations had been made to gain time in handling the increased flows: Fort Loudoun from 22,000 to 27,000 cfs; Watts Bar from 60,000 to 70,000 cfs; Chickamauga from 115,000 to 135,000 cfs; Guntersville from 165,000 to 200,000 cfs; Wheeler from 180,000 to 200,000 cfs; and Pickwick from 195,000 to 210,000 cfs. In anticipation of Cairo exceeding the 40-foot stage, Kentucky discharge was increased to 260,000 cfs to start the drawdown of the reservoir in preparation for storing to reduce the Cairo crest if necessary. The U. S. Weather Bureau was notified of resulting anticipated stages for release to the public.

The first reservoir routing computations for determining the ultimate discharges at mainstream dams were based upon the estimated runoff from the observed rainfall to 6:00 a.m. Saturday, January 18, plus 1.00 inch of predicted rainfall continuing from that hour. These computations indicated that increases of discharges in addition to those earlier increases at mainstream dams would be necessary to provide storage space for regulation of the flood to bankfull or so-called "flood stage" of 30 feet at Chattanooga (fig. 138). Fort Loudoun was increased to 40,000 cfs, Watts Bar to 120,000 cfs, Chickamauga to 155,000 cfs, Guntersville continued at 200,000 cfs, Wheeler increased to 240,000 cfs, Wilson limited to 260,000 cfs, and Pickwick and Kentucky increased to 260,000 cfs. With the changing headwaters it was always necessary to specify that gates at all dams should be adjusted as needed to maintain the discharges within 10,000 cfs of that specified, except at Chickamauga, where the deviation was limited to 5000 cfs. Again, anticipated time and heights of crests at all important points were furnished to the U. S. Weather Bureau for distribution to the public. Developments thus far verified the need for the operating method pursued in maintaining storage space in mainstream reservoirs.

The computed average natural flows for January 17 at Chickamauga and Kentucky Dams were 149,000 and 113,000 cfs, respectively. The computed average system inflows for January 17 at Chickamauga and Kentucky Dams were 206,000 and 366,000 cfs, respectively.

The Regular Forecast issued before noon Saturday, January 18, predicted:

> . . . Precipitation amounts to average . . . 1.00 to 1.50 inches over the Southwestern Section, southern half of the Western Section, western half of the Southeastern Section, and southern half of the

East Central Section, 0.90 to 1.20 inches over the West Central and Eastern Sections, northern half of the East Central Section, eastern half of the Southeastern Section, and northern half of the Western Section, and 0.60 to 0.90 of an inch over the Northeastern Section. Monday and Tuesday and Wednesday: Intermittent periods of rain . . . to average over 1.25 inches.

A second computation of reservoir routing was made using estimated runoffs from an additional inch of rainfall assumed to begin at 6:00 a.m. Sunday, January 19, to ascertain whether releases at critical points could be regulated by the established discharges with still some reservation of storage in the reservoirs. These previously established releases were verified as shown to the right in figure 138, still reserving storage space for necessary regulation of flood crests.

The evening check-up on rainfall development indicated that about 0.3 of an inch had fallen by 6:00 p.m. above Chattanooga with high spots of 0.9 of an inch, and 0.3 of an inch below Chattanooga with high spots of 0.4 of an inch. It was raining in all areas except the Western and Northeastern Sections at the 6:00 p.m. observation time. The first Supplementary Forecast received Saturday evening predicted:

. . . rain continuing over the eastern third tonight. Further precipitation will develop over the basin Sunday and Sunday night as another wave forms in the Gulf. . . . Precipitation amounts . . . average . . . next 24 hours Western 0.50 to 0.70, Southwestern 0.60 to 0.85, West Central 0.70 to 0.95, East Central 0.85 to 1.15, Northeastern 0.60 to 1.00, Southeastern 1.00 to 1.50 inches. Heaviest . . . along the southern edge of the East Central and Southeastern Sections. A few high spots up to 1.75 (inches) . . . there.

The final Supplementary Forecast indicated that the lull in precipitation over the southwestern limits might extend until Sunday morning. No adjustments in releases seemed advisable that night in consideration of the observed rainfall, reservoir conditions, and weather forecast.

January 19 (Sunday)—Rainfall for the 24-hour period ending at 6:00 a.m. Sunday, January 19, averaged about 0.6 inch and 0.3 inch above and below Chattanooga, respectively, with high spots above and below of about 1.2 inches. Rain was continuing at observation time over practically the entire basin.

Sunday morning, January 19, the Weather Bureau telephoned that heavy rains were expected with no end in sight, and that precipitation amounts in the next 24 hours would average about 1.00 inch and 0.50 to 0.75 of an inch in the succeeding 24 hours. Preliminary review of the reservoir system condition resulted in decisions to reduce the Watts Bar spillage from 80,000 to 60,000 cfs to prevent the headwater elevation from dropping below elevation 735, because of certain upstream limitations, and to continue the then existing releases at other dams.

The first mainstream reservoir routing computations were based upon the estimated runoff from the observed rainfall plus an anticipated additional inch beginning at noon in the western half and at 6:00 p.m. in the eastern half. An alternate computation was made for Guntersville, Wheeler, and Pickwick Reservoirs using estimated runoff from still an additional 0.75 inch of rainfall beginning at noon Monday, January 20. It was assumed in this computation that the established discharges would be continued to determine what effect such operation would have upon those reservoirs should the rainfall anticipated in the succeeding 24 hours develop. This alternate routing computation indicated that there would be only a slight reduction in available storage space in Guntersville, but a serious lessening of available storage space would result in Wheeler and Pickwick Reservoirs; therefore, increases should be made in those two reservoirs.

As a result of these two concurrent computations, the following operations were adopted as the most feasible under the unsettled weather conditions prevailing: reduction should be made at Fort Loudoun from 42,000 to 34,000 cfs at noon to maintain the required minimum elevation of 807 for navigation at Knoxville in the Fort Loudoun pool; the earlier reduction at Watts Bar was justified to maintain the minimum elevation 735; the 155,000 cfs at Chickamauga would continue drawing the reservoir and with the local inflow between Chickamauga Dam and Chattanooga anticipated at that time would hold the level at Chattanooga to 30-foot stage, above which it is undesirable to go; the 200,000 cfs at Guntersville would continue drawing that reservoir; Wheeler would have to be increased from 240,000 to 250,000 cfs and Pickwick from 260,000 to 280,000 cfs; Kentucky continued at 260,000 cfs. Changes in discharge were ordered and the Weather Bureau was again advised of anticipated times and heights of crest stages at important points. Tributary reservoirs continued to store water while supplying turbine requirements.

The computed average natural flows at Chickamauga and Kentucky Dams for January 18 were 176,000 and 135,000 cfs, respectively. The average system inflows for January 18 at Chickamauga and Kentucky Dams were 230,000 and 429,000 cfs, respectively. The Cairo observed stage at 6:00 a.m., January 19, was 33.5 feet. No predictions were furnished by the U. S. Weather Bureau at Cairo, Illinois, on Sunday, as no critical stages were in sight on the Ohio and Mississippi Rivers.

The Regular Forecast received about 11:00 a.m. Sunday predicted:

Precipitation amounts . . . to average . . . Southeastern Section 1.40 to 1.80 inches; Eastern, East

> Central, Southwestern, and southern portion of the West Central Section 1.00 to 1.40 inches; Western, Northeastern, and northern portion of the West Central Section 0.80 to 1.20 (inches). Tuesday, Wednesday, and Thursday heavy precipitation.

As these amounts were in substantial agreement with the earlier forecast, the scheduled operations were maintained.

The Weather Bureau advised by telephone on Sunday afternoon that weather conditions appeared favorable for heavy rain by Monday morning. However, there was some indication that the heavy rain, in the order of 2 to 3 inches, might fall Sunday night. Were the movement faster than anticipated, amounts of precipitation would be less. Better information would be made available that evening.

The evening check-up on rainfall development indicated that 0.3 of an inch had fallen since 6:00 a.m. above Chattanooga with high spots of 0.9 of an inch, and 0.4 of an inch below Chattanooga with high spots of 0.8 of an inch. It was raining at the 6:00 p.m. observation time over all the area except the extreme eastern limits. The Supplementary Forecast received Sunday night predicted:

> . . . rain tonight and Monday. Heaviest amounts . . . Monday morning . . . averages . . . East and East Central Sections 1.50 to 2.00 inches, Southwestern Section and southern portion of East Central, Southeastern, and Eastern Sections . . . 2.00 to 2.50 inches with possible high spots of 3.00 inches . . . in thunderstorm areas, Western, West Central, and Northeastern Sections 0.80 to 1.00 inches.

This forecast was amended by a later verbal forecast of:

> Rain tonight ending over the western limits during the morning and over the central portions about 2:00 p.m. (Monday) and over most of the area by late Monday night. Amounts expected to vary from near 0.50 of an inch over the Western Section, 0.75 to 2.50 over the Southwestern Section with heaviest amounts on east end, near 2.00 to 2.50 inches over the Southeastern and East Central Sections, 0.75 to 2.00 over the West Central with heaviest amounts over the east end, and 1.00 to 1.50 inches over the Eastern and Northeastern Sections.

This amended forecast indicated that the storm was reaching its climax and would be over by Monday night.

Storing for peak reduction

Now the operation of the system could be shifted from emphasis on the reservation of storage space to the utilization of some of that space for regulation of flood flows. That night (Sunday, January 19) routing computations were made based upon estimated runoff from rainfall observed to 6:00 p.m., plus allowance for the additional anticipated amounts and upon the principle of then storing to effect a reduction in the natural flood crests at key points, yet reserving some storage space as a margin of safety in the event of possible additional rainfall later.

These computations indicated a slight increase necessary in Fort Loudoun discharge from 34,000 to 40,000 cfs while storing to elevation 810.1; maintaining 100,000 cfs at Watts Bar while storing to elevation 738.3; maintaining the 155,000 cfs at Chickamauga while storing to elevation 678.8 and attempting to regulate the stage at Chattanooga to 32 feet (at this stage there would be little or no damage as it is only 2 feet above flood stage and materially lower than the anticipated natural crest); increasing Guntersville to 225,000 cfs while storing to elevation 594.0; increasing Wheeler to 300,000 while storing to elevation 552.8; increasing Pickwick to 325,000 while storing to elevation 416.1; and increasing Kentucky to 280,000 cfs in order to draw the reservoir about ½ foot to 1 foot per day. These increases in discharges were authorized to be made about 11:00 p.m. Sunday night.

While these adjustments were being made to the main river reservoirs the tributary reservoirs continued storing while supplying turbine requirements, and advisory warnings were issued again to the Weather Bureau for broadcasting to the public.

January 20 (Monday)—The rainfall observed for the 24-hour period ending at 6:00 a.m., Monday, January 20, averaged about 1.9 inches above Chattanooga and about 1.1 inches below, with high spots of 3.7 inches above and 2.3 inches below. Rain was continuing at observation time over the eastern half and at scattered points in the western half of the Basin.

The Preliminary Forecast received Monday, January 20, predicted:

> . . . rain over the western half, ending this morning and . . . ending over the southern limits of the eastern half of the area during the morning and northeastern limits late this afternoon. . . . Precipitation . . . average . . . 0.60 to 0.80 of an inch over the East Central, Southeastern, and the Eastern Sections . . . 0.80 to 1.00 over the Northeastern Section, 0.40 to 0.60 inch over the eastern half of the West Central and Southwestern Sections, and from a trace to 0.40 of an inch elsewhere.

The Weather Bureau telephoned at 9:30 a.m. to amend this Preliminary Forecast to the extent that the rain was over except for a few scattered showers.

Therefore, the routing computations were based upon the estimated runoff from the observed rainfall, which was somewhat heavier than predicted on mainstream reservoirs above Chattanooga and slightly lighter on those below. The computed average natural flow at Chickamauga Dam for Sunday, January 19, was 218,000 cfs, while the average system inflow above Chickamauga Dam for that date was 223,000 cfs. A preliminary estimate of anticipated natural flow crest at Chickamauga Dam indicated it would be about 295,000 cfs on Wednesday, January 22. The objective was to reduce the natural flow crest as much as possible at each point and, as soon as possible after the flood crest passed, to get rid of the surplus water stored in the mainstream reservoirs in order to return them to normal flood season levels before releasing water stored in the tributary reservoirs. This operation necessitated increasing the discharge at Fort Loudoun to 50,000 cfs and at Watts Bar to 120,000 cfs. Chickamauga discharge was maintained at 155,000 cfs to minimize the flood damages at Chattanooga, which were very small, by temporarily continuing to store in Chickamauga Reservoir.

In the routing computations it was found that spillage could be started from tributary reservoirs on Wednesday, January 22, without sustaining existing water surface profiles below Chickamauga. Because of the time of water travel from the tributary reservoirs to mainstream reservoirs, the increased discharges would not arrive fully until January 23. With the discharge from Chickamauga Reservoir being held down to 155,000 cfs it was possible to make reductions at Guntersville to 200,000 cfs, Wheeler to 250,000 cfs, and Pickwick to 275,000 cfs. These reductions for the benefit of downstream points caused a temporary storing in those reservoirs.

Drawdown of Kentucky Reservoir for Lower Ohio and Mississippi regulation

The computed average natural flow for January 19 at Kentucky Dam was 162,000 cfs. The 6:00 a.m. observed stage at Cairo for January 20 was 33.5 feet with predictions for the three succeeding days of 35.9, 37.9, and 39.2 feet with crest prediction of 40.5 feet for Saturday, January 25. In view of this definite crest prediction, Kentucky discharge was increased to 300,000 cfs in order to accelerate the drawdown of the reservoir in advance of the Ohio River crest so that storage space would be maintained for reducing that crest, or a higher crest which might occur later with additional rain, on the Ohio and Mississippi Rivers. This discharge at Kentucky was based upon a preliminary estimate of anticipated natural flow crest at Kentucky Dam which indicated it would be slightly greater than 300,000 cfs.

The Regular Forecast received in the forenoon Monday, January 20, predicted only a few scattered showers accompanying a cold front, with amounts expected to average from 0.20 to 0.40 of an inch in the eastern half and less than 0.20 west of a line between Chattanooga and Knoxville. The weather outlook for the rest of the week indicated no significant precipitation. The succeeding days of fair weather permitted further reductions in releases from Pickwick and upper mainstream reservoirs which were lowered to about normal flood season levels by the end of January. Kentucky Reservoir was drawn to elevation 349.2 by January 22. At that time, it became apparent that the stage at Cairo would not exceed 40 feet materially, and Kentucky Reservoir was gradually refilled to about the normal flood season level (elevation 354) by the end of the month.

In view of this relatively low prospective crest stage at Cairo, the exchange of data between TVA and the Corps of Engineers was not required by the Ohio River Division Engineer at Cincinnati.

Release of stored water after crest

Spilling of stored floodwater began at Norris, Cherokee, and Douglas Reservoirs on January 22, and at Hiwassee and Fontana Reservoirs on January 23. Nottely Reservoir spillage was begun on January 24. Releases were increased from day to day to about tributary bankfull stage as mainstream flows receded. Cherokee, Hiwassee, and Fontana were returned to about maximum multiple-purpose levels by the end of the month, and Norris, Douglas, and Nottely were being drawn rapidly. Figures 135 and 136, pages 242 and 243, depict chronologically the regulated elevations in the multiple-purpose mainstream and tributary reservoirs during the January 1947 flood period.

Effect of flood control during January 1947

Computations show that under natural flow conditions the stage at Chattanooga would have risen

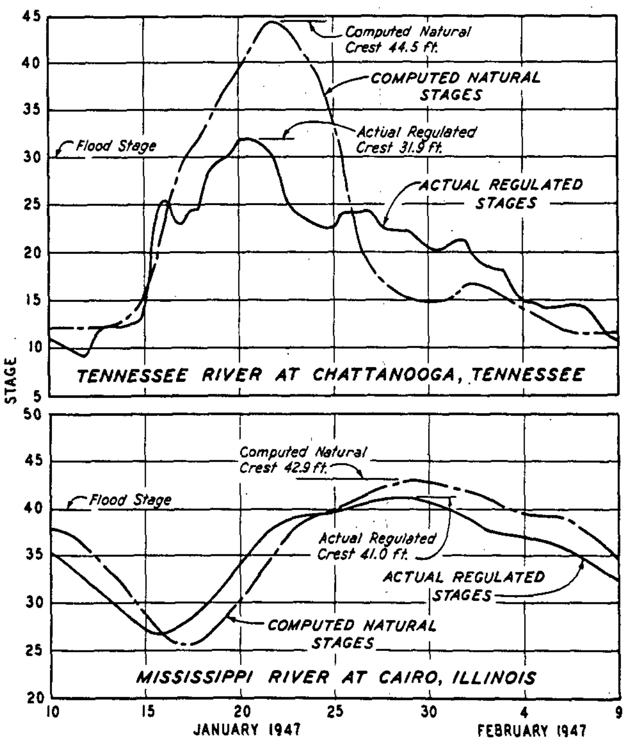


FIGURE 139.—*Effect of reservoir operation on flood stages at Chattanooga and Cairo—January-February 1947.*

to 44.5 feet, the sixth highest flood of record up to that date. Physical damages from such a flood would have amounted to about $11,500,000. By regulating the stage at Chattanooga to 31.9 feet, practically this entire damage was prevented.

Downstream regulation combined with the advisory warnings broadcast by the U. S. Weather Bureau confined the damages largely to the inconvenience from minor flooding, because time was available to move property and livestock from low areas. Computations indicate that without the benefit of the TVA multiple-purpose reservoir system the stage at Cairo would have been about 1.9 feet higher than the observed regulated stage of 41.0 feet. Figure 139 indicates the effect of the regulation by the system of reservoirs on stages at Chattanooga and Cairo compared with the stages which would have occurred without regulation. Figure 140 gives a comparison of regulated and natural flows at Chickamauga and Kentucky Dams with the corresponding system inflow.

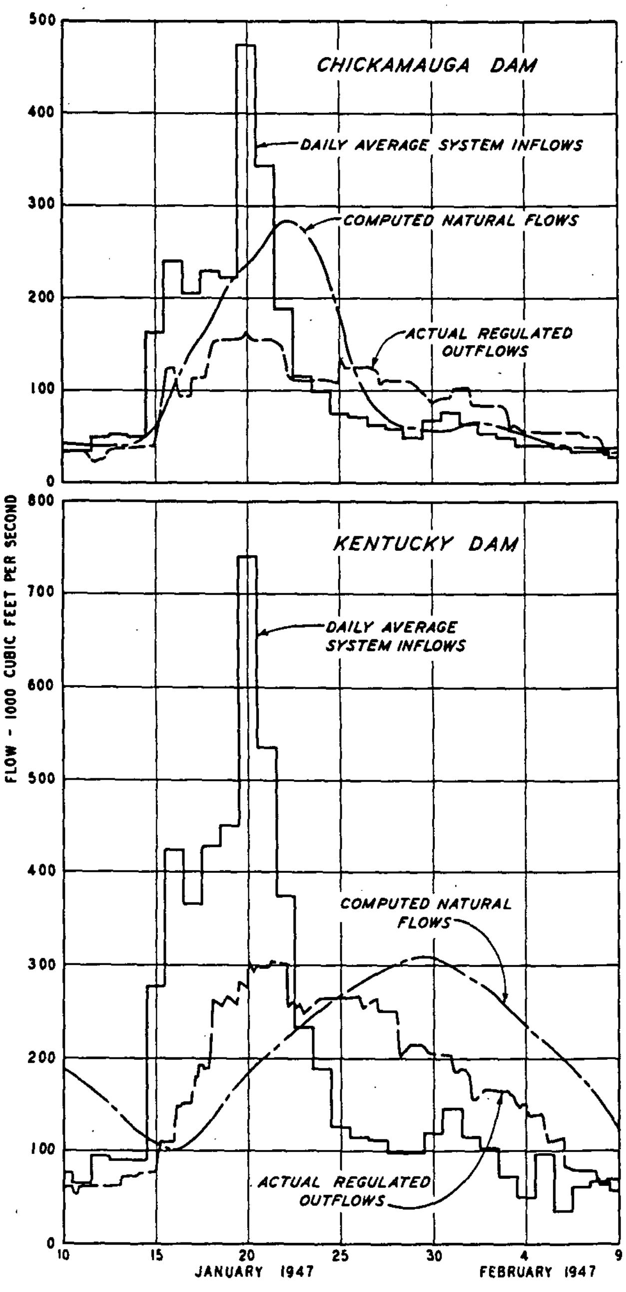


FIGURE 140.—*Comparison of regulated and natural flood flows at Chickamauga and Kentucky Dams—January-February 1947.*

Reductions in stages were also effected at other points, both above and below Chattanooga, as well as below each tributary dam. The stage at Knoxville was reduced about 9.3 feet below the uncontrolled or natural stage; at Florence it was reduced about 2.8 feet and at Savannah about 7.7 feet. Figure 141 illustrates the reductions in flows effected below several dams during this flood.

The regulation of the January 1947 flood was effectual, and it also exemplifies the coordination between water control operations and U. S. Weather Bureau forecasts as then in effect. Especially important in the successful regulation of this flood was the inclusion of at least a portion of rainfall predicted from time to time by the Weather Bureau in computing flows and determining necessary discharges. Whenever practicable, this is done.

RESULTS OF FLOOD REGULATION JANUARY 1946

Also noteworthy is the successful regulation of the even higher January 1946 flood (fig. 134, page 240). Figure 142 depicts the drawdown and subsequent storing in Kentucky Reservoir to reduce the lower Ohio and Mississippi River stages, resulting reductions at Paducah and Cairo, and comparison of actual and natural discharges from Kentucky Reservoir during the 1946 flood. These operations resulted in reducing relatively high crests at Paducah and Cairo to stages 45.9 and 52.1 feet, reductions of 2.3 and 1.4 feet, respectively, while storing only 5.3 feet above minimum flat pool level in the reservoir. This regulation also follows quite well the basic plan of operating this reservoir.

A number of features concerning this 1946 operation in Kentucky Reservoir are worthy of note. As the inflow increases, a considerable volume of water goes into profile storage in the head of the reservoir. Consequently, if the rate of spill at the dam is made approximately equal to the inflow, a volume equivalent to the profile storage is withdrawn from the lower end of the reservoir. However, to accomplish this successfully, the spill must be started while the inflow is low and increased with it; otherwise the operation would involve sudden dumping at high rates into the river below the dam with accompanying undesirable results.

Although this operation may not be essential from the standpoint of preserving space for storing moderate floods, there is no way at this season of the year to predict how large a flood may actually develop and, as pointed out above, the operation must be started in the early stages to be successful. In addition to preserving storage space, this operation also makes it possible to pass considerable quantities of water into the Ohio ahead of the crest on that stream.

The rise in reservoir surface shown in figure 142, subsequent to the drawdown, does not necessarily signify impoundment of a greater volume of water in the reservoir, but rather a redistribution of the profile storage as the flow through the reservoir decreases. This phenomenon occurs automatically during recession, and were it not for the additional space provided previously by drawdown, either the headwater at the dam would rise to higher levels or the high rate of spill would have to be continued for a longer period. Either one of these alternatives could be undesirable.

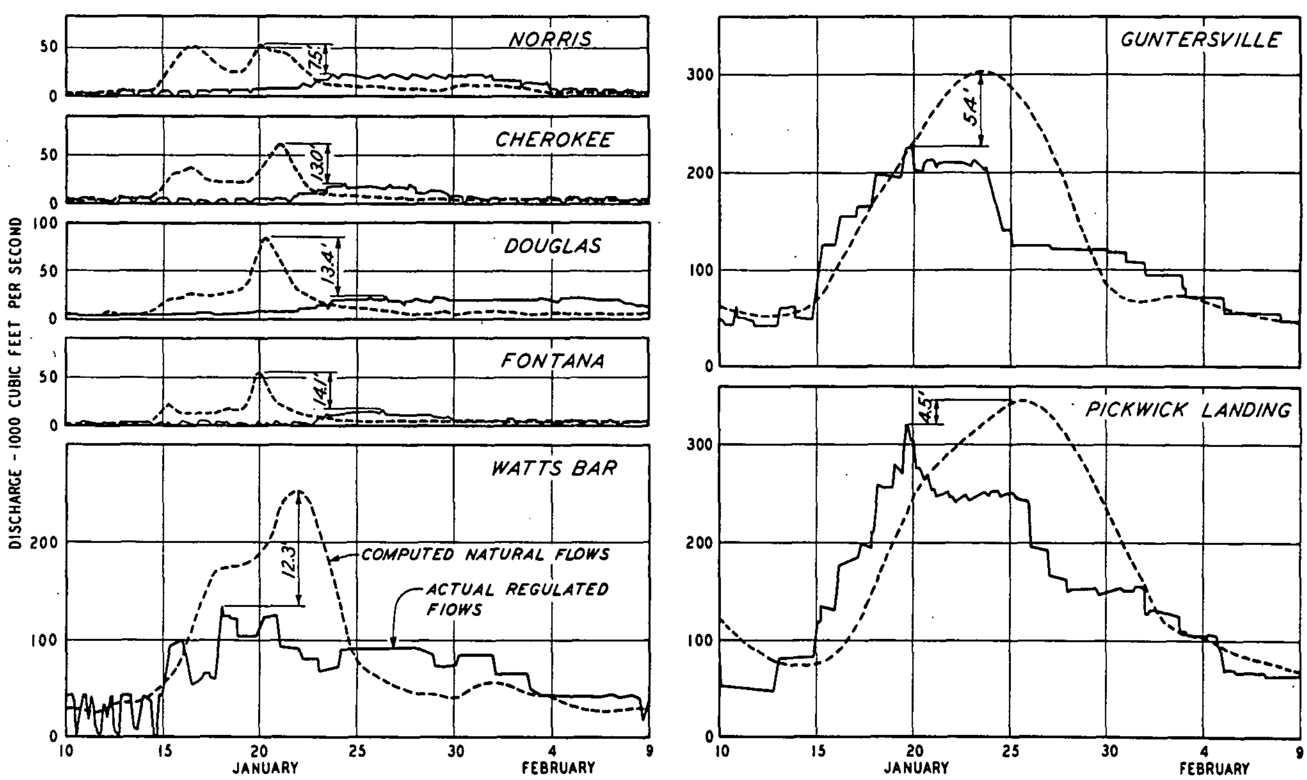


FIGURE 141.—*Reduction of January 1947 flood below four tributary and three main-river dams.*

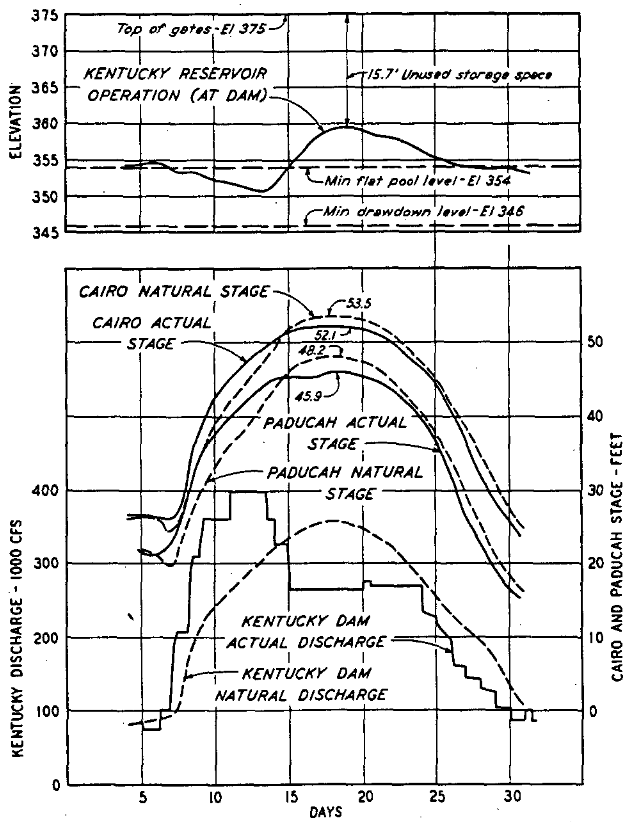


FIGURE 142.—*Regulation of January 1946 flood crests at Paducah and Cairo by Kentucky Reservoir operation.*

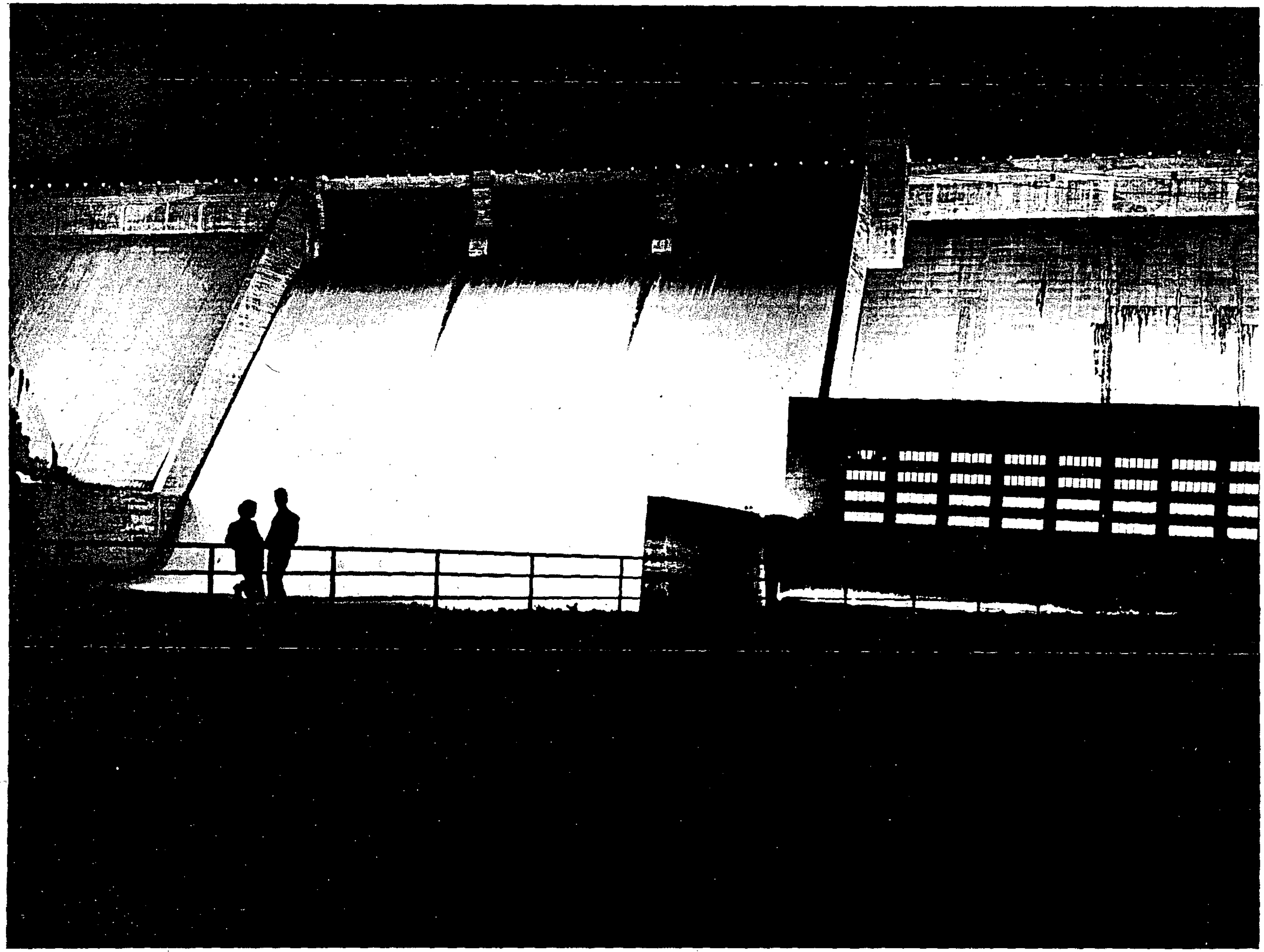


FIGURE 143.—*Norris Dam releasing water impounded during a flood control operation.*

APPENDIX B

POSSIBLE FLOOD CREST REDUCTIONS

This appendix presents numerical and graphical data concerning Tennessee Valley pre-reservoir and post-reservoir floods, these data resulting principally from the application of methods and principles described or outlined in the body of the report. Among these data are hydrographs (fig. 144) showing operation of the system reservoirs during the 1950 flood—one which tested the system. The discussions cover methods of computing flood reductions, and results of actual operation—including a comparison with planning studies—of the reservoirs for flood control. A brief discussion of the operation of proposed detention basins above Asheville on the French Broad River concludes the appendix.

Tables 50 through 56 give the maximum discharges and stages below TVA dams and at other important locations for the Chattanooga maximum probable flood; an approximation of the maximum known flood on the Tennessee River above the mouth of the Elk River—1867; the maximum known flood on the lower Tennessee River—1897; the maximum known flood on the lower Ohio and Mississippi Rivers—1937; and for three post-reservoir floods—1946, 1950, and 1957.

TABLE 50.—*Peak discharges and stages below TVA dams and at other important locations, natural and fixed-rule operation Chattanooga maximum probable flood.*

Location	Computed natural[1] Discharge, cubic feet per second	Computed natural[1] Stage, feet	Computed fixed rule[3] Discharge, cubic feet per second	Computed fixed rule[3] Stage, feet
Cherokee Dam	95,000	949.8	50,000	939.5
Douglas Dam	161,000	901.3	100,000	893.2
Knoxville[2]	286,000	842.2	163,000	830.3
Fort Loudoun Dam	293,000	787.9	176,000	775.1
Fontana Dam	100,000	—	57,000	—
Norris Dam	138,000	857.1	39,000	836.0
Watts Bar Dam	615,000	726.7	386,000	711.0
Hiwassee Dam	70,000	1292.5	22,000	1281.8
Chickamauga Dam	716,000	700.3	476,000	684.4
Chattanooga	730,000	77.2	497,000[4]	60.9[4]

1. Tributary and local components of the Chattanooga Design Flood were determined from rainfall and runoff of the 1936 flood. Time was compressed and runoff expanded to give a natural peak of 730,000 cubic feet per second on March 15 at Chattanooga.
2. At site of Old Water Plant gage, mile 648.16.
3. As regulated by the present-day system with the exception of South Holston, Watauga, and Boone Dams.
4. 486,000 cubic feet per second and 60.0 feet were used for the design of the Chattanooga levee system, as shown in the report, "The Chattanooga Flood Control Problem," 1939.

Figure 144 shows discharges and stages for computed natural, actual regulated, and computed fixed-rule conditions for the flood of 1950 at major tributary dams and all main-river dams and for other critical points. This flood was the largest on the Mississippi River since completion of the TVA system. It is presented as an example of a flood which tests the system.

Natural crests for post-reservoir floods were computed by routing inflows—determined from the actual operation—downstream using natural routing curves described in chapter 6. Discharges and stages for the fixed-rule operation were computed by methods described.

Tables 51 and 52 also show elevations of maximum known floods—those of 1867 and 1897. For natural conditions, these elevations were determined from high water mark profiles and for regulated conditions from a conservative estimate of the reduction provided by the reservoir system.

Because there are no gaging station records for the 1867 flood, the 1875 flood, increased by 10 percent, has been used as representative. This agrees with the known 1867 crest stage at the most critical place, Chattanooga, but differs at a number of other places.

METHODS OF COMPUTING FLOOD REDUCTIONS

Before making reservoir operation studies of pre-reservoir floods, two factors had to be determined. One of these, storage in the various reservoirs or natural river reaches, depended on the characteristics of the river channel; and the other, inflow, depended on characteristics of the particular storm.[1]

Tributary reservoir volume

High dams built on the tributaries, combined with the steep slope of the natural streams, produce an almost level pool, there being a significant backwater curve only in a relatively short distance at the upper end of the pool, where the natural depth and slope of the stream are soon reached. The volume

1. The general methods used in the TVA studies for the evaluation of these two factors have been described in detail in the *Transactions, American Society of Civil Engineers*, Vol. 104, 1939, pages 275-313, and in chapter 6 of this volume.

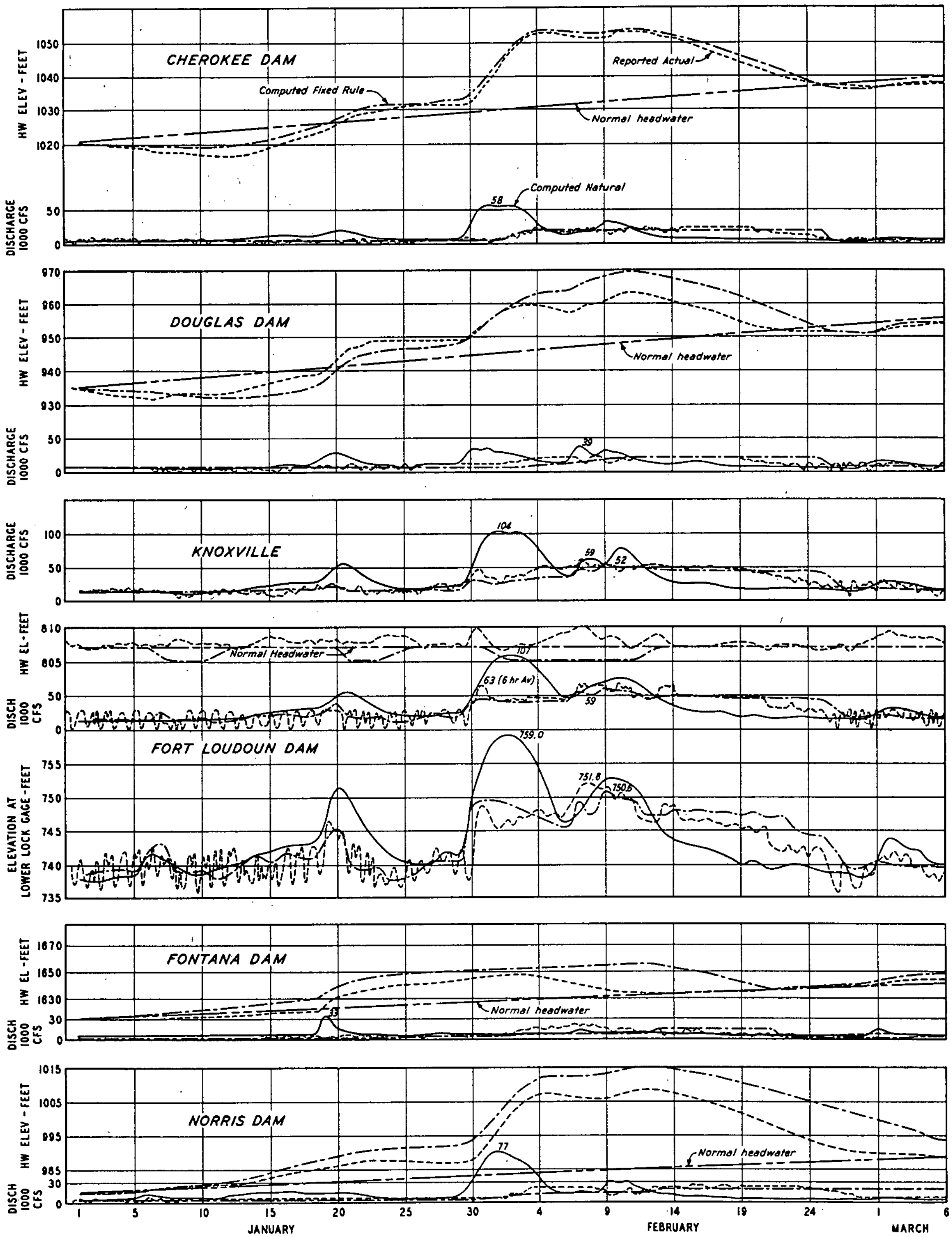


FIGURE 144.—*(sheet 1 of 5)—Reservoir operation—1950 flood.*

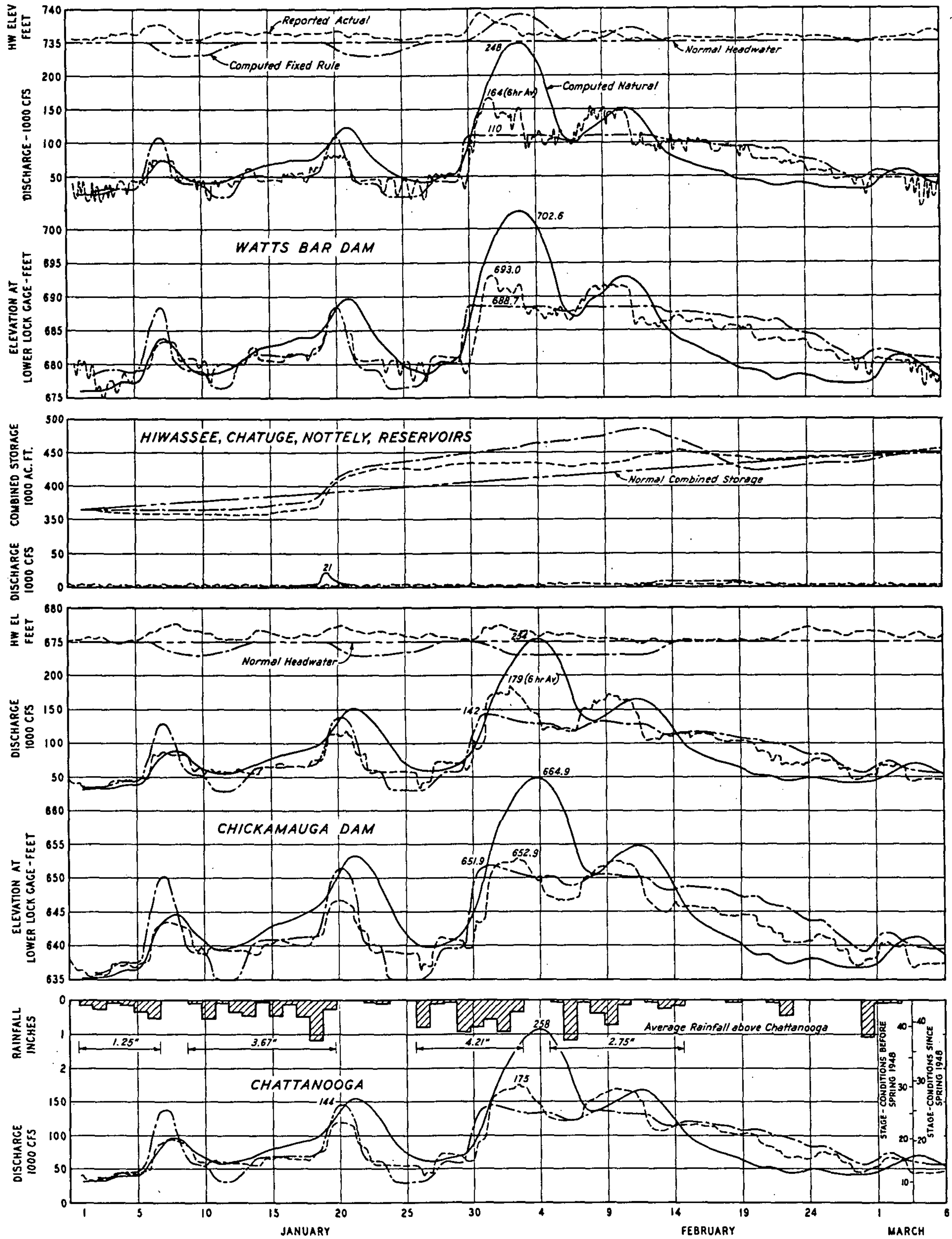


FIGURE 144.—*(sheet 2 of 5)—Reservoir operation—1950 flood.*

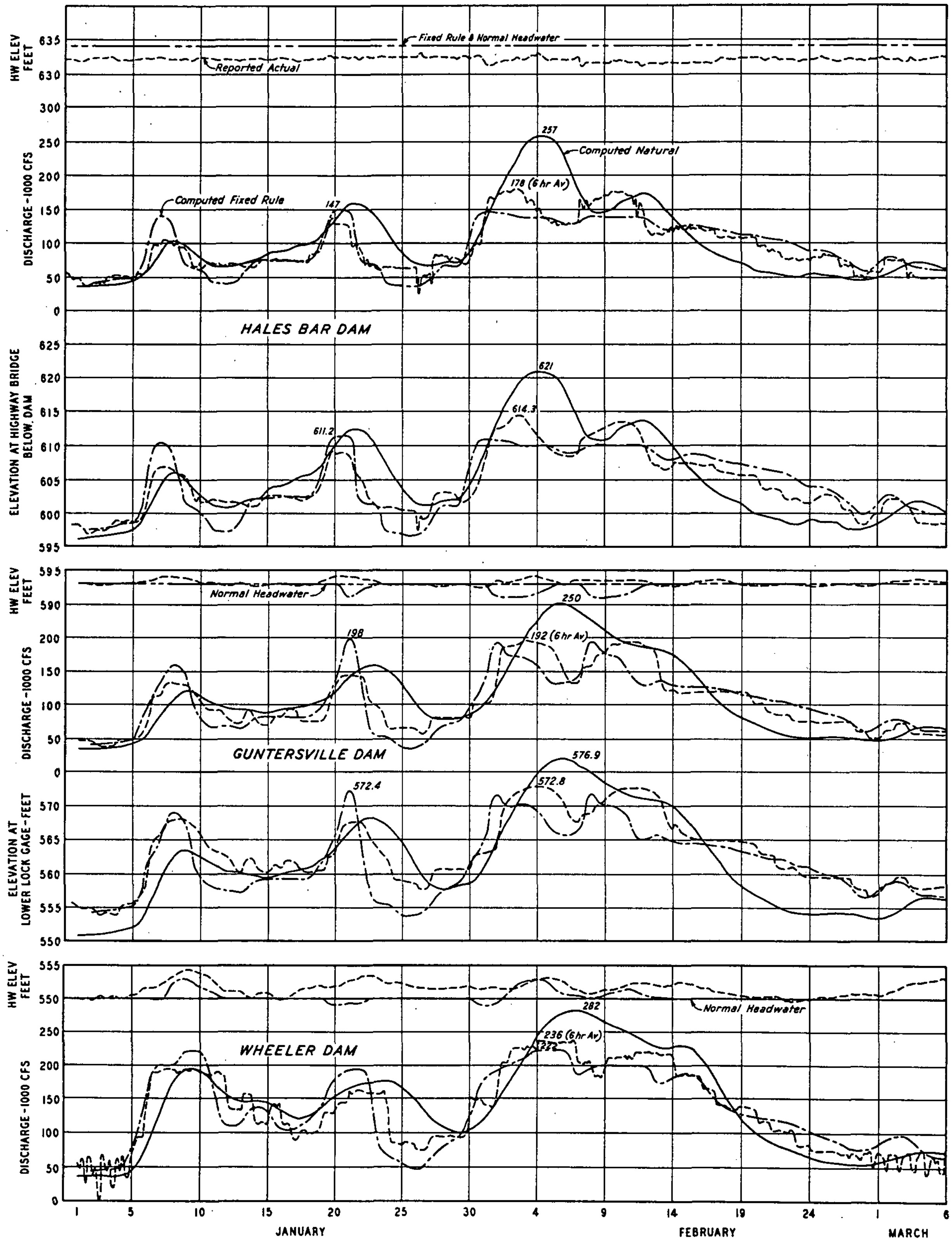


FIGURE 144.—*(sheet 3 of 5)—Reservoir operation—1950 flood.*

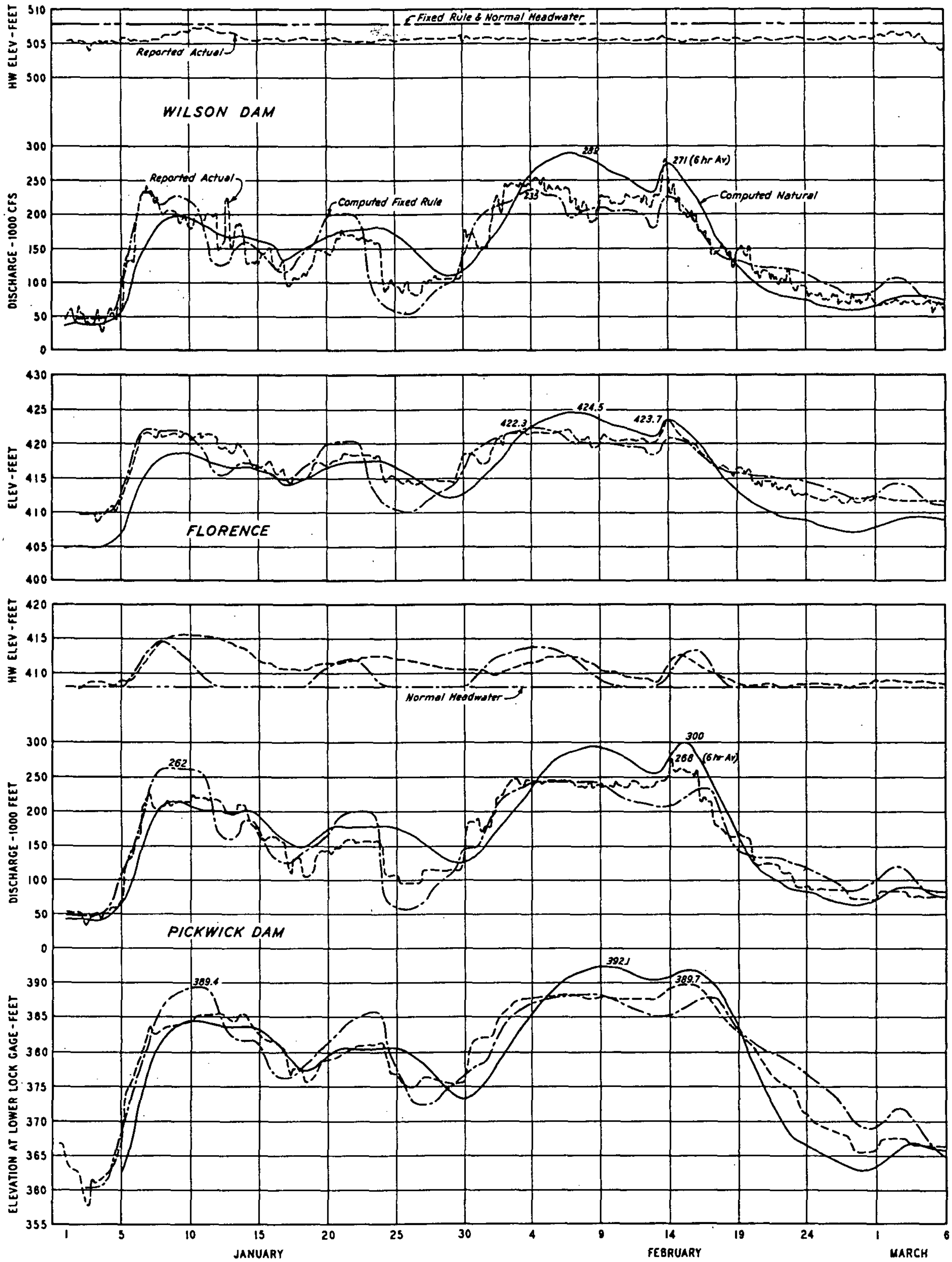


FIGURE 144.—*(sheet 4 of 5)—Reservoir operation—1950 flood.*

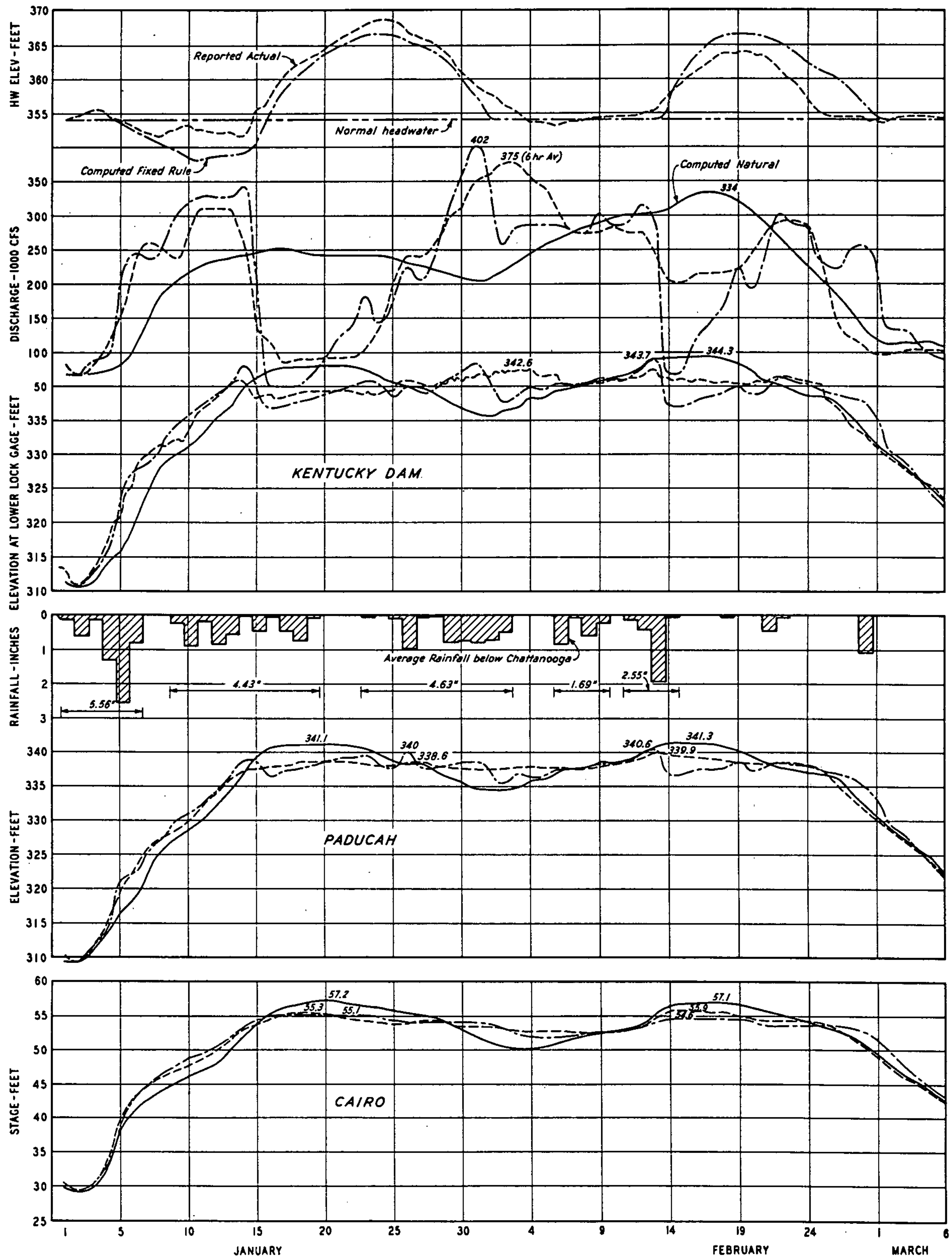


FIGURE 144.—*(sheet 5 of 5)—Reservoir operation—1950 flood.*

TABLE 51.—*Peak discharges and stages below TVA dams and at other important locations, natural and fixed-rule operation March 1867 flood.*

Location	Computed natural[5] Discharge, cubic feet per second	Computed natural[5] Stage, feet	Profile of natural maximum known flood stage, feet	Computed fixed rule[6] Discharge, cubic feet per second	Computed fixed rule[6] Stage, feet	Profile of regulated maximum known flood stage, feet
Cherokee Dam	116,000	953.4	959.8	44,000	938.0	938.0
Douglas Dam	150,000	900.0	900.0	60,000	886.6	886.6
Knoxville[1]	315,000	845.3	842.7	120,000	825.0	823.9
Fort Loudoun Dam	315,000	784.5	787.4	125,000	763.2	764.9
Fontana Dam	65,000	—	—	20,000	—	—
Norris Dam	75,000	844.1	—	15,000	829.1	—
Watts Bar Dam	463,000	718.5	717.6	216,000	698.1	702.4
Hiwassee Dam	26,000	1281.5	—	14,000	1280.1	—
Chickamauga Dam	446,000	681.5	682.4	244,000	661.6	664.8
Chattanooga	459,000	57.9	57.9	247,000	37.6	39.8
Hales Bar Dam[2]	460,000	634.1	634.1	249,000	620.2	623.2
Guntersville Dam	419,000	587.8	590.1	268,000	578.3	583.3
Wheeler Dam	429,000	507.3	—	311,000	508.5	—
Wilson Dam	442,000	—	—	332,000	—	—
Florence	—	432.1	—	—	426.8	—
Pickwick Dam	476,000	403.6	—	375,000	396.8	—
Savannah	—	400.1	—	—	394.3	—
Kentucky Dam[3]	424,000	345.1	—	365,000	342.6	—
Paducah[3, 4]	—	340.0	—	—	338.8	—
Cairo[3]	—	58.2	—	—	55.7	—

1. At site of Old Water Plant gage, mile 648.16.
2. At USGS gage, Highway Bridge, mile 429.7.
3. Present Mississippi and Ohio River channel conditions as completely confined within levees. Natural Cairo stage would be about 57.5 feet if the Birds Point-New Madrid Floodway began to operate at about 56.5 feet. In 1867 actual Kentucky tailwater was 342.8 (from profile); Paducah elevation and Cairo stage, 337.9 and 51.0 feet (from gage readings).
4. USGS gage at foot of Jefferson Street.
5. Because of the complete lack of gaging station records during 1867 in the Tennessee River Basin, the 1875 flood plus 10 percent has been used as representative. This agrees with the known 1867 crest at the most critical place, Chattanooga, but differs at a number of other places.
6. As regulated by the present-day system with the exception of South Holston, Watauga, and Boone Dams.

in these tributary reservoirs at any instant was considered to be that under a level line corresponding to the headwater level at the dam. These volumes were determined by planimetering contours shown on topographic maps.

Tennessee River reservoir volume

Water surface slope of the Tennessee River, however, is considerable during flood periods even with the reservoirs. Except for relatively minor changes caused by improved channel carrying capacity, the water level at the head of each reservoir is at least as high for the same flow as it was under natural conditions. Storage under this slope is a significant amount and cannot be neglected. This storage, combined with storage under the flat pool corresponding to the headwater level at the downstream dam, was related to the total inflow into the reservoir, to the total outflow at the downstream dam, and to the headwater elevation.

Storage under profiles of equal inflow-outflow was obtained in the same manner as described under the heading "Computation of natural storage," chapter 6, page 123. A greater number of profiles were necessary, of course, because of the variable headwater levels at the downstream dam. A series of storage values was computed on this basis for assumed inflows, outflows, and headwater elevations for each of the Tennessee River reservoirs from Fort Loudoun to Pickwick Landing. The family of curves in figure 145 shows the relation between these four variables for Chickamauga Reservoir. Charts for the other reservoirs are similar.

The method used to determine storage for flood routing in Kentucky Reservoir differed from that used for the other Tennessee River reservoirs. Here, because of its great length (184 miles), the storage under a profile cannot be defined by inflow, outflow, and headwater elevation. After a number of trials, storage curves were computed by taking the summation of short-reach volumes under backwater profiles for steady flow throughout the reservoir and for various headwater levels at Kentucky Dam. A family of curves was constructed showing the relation for uniform flows between Pickwick tailwater elevation, Kentucky headwater elevation, and storage. To determine storage for non-steady flow profiles, it was necessary to make an adjustment, and it was found that this could be made to the best advantage in Pickwick tailwater. This adjustment was developed

TABLE 52.—*Peak discharges and stages below TVA dams and at other important locations, natural and fixed-rule operation March-April 1897 flood.*[1]

Location	Natural: Discharge, cubic feet per second	Natural: Stage, feet	Computed fixed rule[6]: Discharge, cubic feet per second	Computed fixed rule[6]: Stage, feet	Profile of regulated maximum known flood stage, feet
Cherokee Dam	49,000	939.2	22,000	931.2	—
Douglas Dam	63,000	887.1	26,000	878.8	—
Knoxville[2]	137,200	824.5	54,000	813.3	—
Fort Loudoun Dam	136,000	763.1	58,000	752.9	—
Fontana Dam	38,000	—	21,000	—	—
Norris Dam	80,000	945.3	20,000	830.7	—
Watts Bar Dam	247,000	702.4	126,000	690.0	—
Hiwassee Dam	14,000	1277.5	10,000	1279.8	—
Chickamauga Dam	251,000	663.8	178,000	654.4	—
Chattanooga	258,000	39.6	181,000	30.0	—
Hales Bar Dam[3]	261,000	621.2	186,000	615.0	—
Guntersville Dam	280,000	579.1	214,000	574.9	—
Wheeler Dam	443,000	507.4	346,000	508.7	509.7
Wilson Dam	481,000	—	379,000	—	—
Florence	—	433.6	—	428.5	430.8
Pickwick Dam	498,000	405.5	431,000	399.4	401.6
Savannah	—	401.0	—	396.9	397.6
Kentucky Dam[5]	471,000	346.0	315,000	340.3	—
Paducah[4, 5]	—	339.8	—	337.1	—
Cairo[5]	—	59.2	—	56.7	—

1. This flood at and above Guntersville was a series of several crests. The table shows only the highest at each place regardless of when it occurred.
2. At site of Old Water Plant gage, mile 648.16.
3. At USGS gage, Highway Bridge, mile 429.7.
4. Foot of Broadway Street gage.
5. Present Mississippi and Ohio River channel conditions as completely confined within levees. Natural Cairo stage would be 58.1 feet if the Birds Point-New Madrid Floodway began to operate at about 56.5 feet. In 1897, the actual peak Kentucky flow was 475,000 cubic feet per second; Kentucky tailwater, 344.1 (from profile); Paducah elevation and Cairo stage, 336.9 and 51.7 feet (from gage readings).
6. As regulated by the present-day system with the exception of South Holston, Watauga, and Boone Dams.

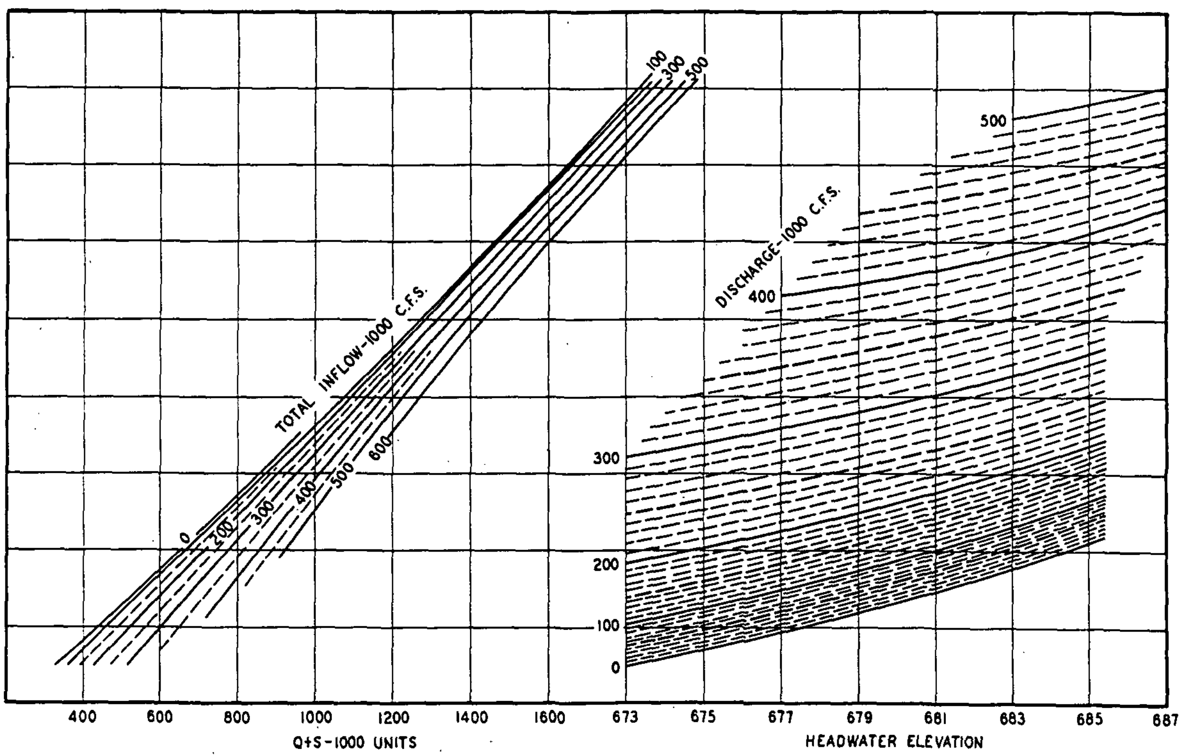


FIGURE 145.—*Regulated routing curve—Chickamauga Reservoir.*

TABLE 53.—*Peak discharges and stages below TVA dams and at other important locations, natural, observed, and fixed-rule operation January-February 1937 flood.*

	Computed natural		Observed[1]		Computed fixed rule[2]	
Location	Discharge, cubic feet per second	Stage, feet	Discharge, cubic feet per second	Stage, feet	Discharge, cubic feet per second	Stage, feet
Cherokee Dam	30,100	933.9	—	—	20,000[8]	930.5
Douglas Dam	64,500	887.4	—	—	25,000	878.4
Knoxville[3]	—	—	90,900	816.6	68,000	815.3
Fort Loudoun Dam	92,000	758.1	—	—	60,000	751.9
Fontana Dam	31,600	—	—	—	15,000[8]	—
Norris Dam	48,000	838.1	42,500[8]	836.8	20,000[8]	830.6
Watts Bar Dam	193,000	697.4	159,000[9]	693.9	110,000	688.2
Hiwassee Dam	22,400	1280.5	—	—	10,000[8]	1279.8
Chickamauga Dam	220,000	660.3	192,000[9]	658.0	162,000	653.3
Chattanooga	225,000	36.0	204,000	33.0	172,000	29.0
Hales Bar Dam[4]	228,000	618.6	209,000	617.0	182,000	614.6
Guntersville Dam	228,000	575.0	210,000[9]	574.2	212,000	574.1
Wheeler Dam	247,000	—	238,000[10]	—	248,000	—
Wilson Dam	253,000	—	237,000[10]	—	254,000	—
Florence	—	422.4	—	421.4	—	423.3
Pickwick Dam	273,000	390.7	253,000[11]	389.4	264,000	389.7
Savannah	—	385.6	—	383.7	—	384.7
Kentucky Dam[6]	349,000	348.7	324,000[11]	347.8	361,000	346.4
Paducah[5, 6]	—	347.1	—	346.5	—	345.7
Cairo[6]	—	59.8	—	59.5	—	58.7
Kentucky Dam[7]	349,000	349.9	—	—	393,000	349.0
Paducah[5, 7]	—	348.4	—	—	—	346.6
Cairo[7]	—	63.7	—	—	—	61.2

1. Only Wilson, Hales Bar, Wheeler, and Norris projects were in operation at this time.
2. As regulated by the present-day system with the exception of South Holston, Watauga, and Boone Dams. Stages are from rating curves which represent average conditions.
3. At site of Old Water Plant gage, mile 648.16.
4. At USGS gage, Highway Bridge, mile 429.7.
5. USGS gage at foot of Jefferson Street.
6. The Birds Point-New Madrid Floodway as actually operated.
7. Completely confined within Ohio and Mississippi River levees.
8. This is an emptying flow. The regulated flow was lower at the time of critical downstream flood conditions.
9. As based on nearest gage.
10. Maximum 6-hour average discharge.
11. Computed.

from an analysis of the floods of January and February 1946, January 1947, and February 1948, in which storages under instantaneous observed profiles were determined. By trial it was found that, if actual Pickwick tailwater elevations were adjusted, good agreement would be obtained between storages read from the family of curves (fig. 146) and that determined from instantaneous profiles. This adjustment was made by distributing two-thirds of a rising daily change to the day of occurrence and one-third to the following day, and one-fifth of a falling daily change to the day of occurrence and one-fifth to each of the four succeeding days. Daily summations of these distributed stage changes were then applied to the initial Pickwick tailwater elevation to determine the adjustment tailwater to use with the curve.

All the storage curves described above give the total volume between the profile as defined by inflow, outflow, and headwater elevation and the former natural low-water profile. They are used in reservoir operation studies of pre-reservoir floods to determine outflow, headwater elevation, and storage stage, and in post-reservoir floods to determine inflow, particularly local inflow from areas between dams.

The use of these curves for the main Tennessee River, together with an estimated inflow, assures full accounting of the fact that the reservoirs now occupy some of the channel and overflow area formerly filled by a natural flood as it developed and passed downstream. Under natural river conditions the valley storage is being filled, without artificial control, of course, from the beginning of the rise up to the peak. In other words, valley storage is the flood. At the time of the flood peak, there is no remaining storage available to reduce the flood. Under reservoir conditions on the other hand, as the flood increases, the headwater level at the dam is held down by increasing the discharge over the spillway, even at times to a greater amount than the natural discharge would have been; but when the peak flow arrives, the headwater level at the dam is allowed to rise, and

TABLE 54.—*Peak discharges and stages below TVA dams and at other important locations, natural, observed, and fixed-rule operation January 1946 flood.*[1]

	Computed natural		Observed[2]		Computed fixed rule[3]	
Location	Discharge, cubic feet per second	Stage, feet	Discharge, cubic feet per second	Stage, feet	Discharge, cubic feet per second	Stage, feet
Cherokee Dam	64,800	943.2	21,800[7]	931.3	14,000[7]	928.4
Douglas Dam	95,900	892.5	21,300[7]	877.3	20,000[7]	877.0
Knoxville[4]	153,000	826.6	51,600	814.2	36,000	810.3
Fort Loudoun Dam	138,000	756.4	64,800	753.5	62,000	753.8
Fontana Dam	44,100	—	18,700[7]	—	10,000[7]	—
Norris Dam	77,500	844.8	21,200[7]	831.6	15,000[7]	829.0
Watts Bar Dam	281,000	705.6	163,000	694.7	103,000	687.6
Hiwassee Dam	19,600	—	11,900[7]	—	6,000[7]	—
Chickamauga Dam	301,000	669.8	208,000	660.0	168,000	654.4
Chattanooga	320,000	45.8	225,000	35.7	181,000	30.0
Hales Bar Dam[5]	319,000	625.4	231,000	618.5	195,000	615.9
Guntersville Dam	320,000	581.9	260,000	577.6	225,000	575.6
Wheeler Dam	380,000	—	332,000	—	320,000	—
Wilson Dam	382,000	—	349,000	—	323,000	—
Florence	—	429.3	—	426.2	—	426.5
Pickwick Dam	380,000	397.5	353,000[8]	396.0	328,000	394.8
Savannah	—	394.3	—	391.6	—	390.8
Kentucky Dam	357,000	338.7	400,000[8]	337.9	362,000	336.7
Paducah[6]	—	333.9	—	331.8	—	331.9
Cairo	—	53.5	—	52.1	—	51.1

1. The TVA system was essentially completed at this time. All reservoirs were near normal levels when the flood began. Fixed-rule operations were begun at normal levels.
2. Stages at tributary projects are the higher of twice-daily readings. At main-river dams they are from recorder charts. Reservoir discharges are the highest average 6-hour flows.
3. As regulated by the present-day system with the exception of South Holston, Watauga, and Boone Dams. Stages are from rating curves which represent average conditions.
4. At site of Old Water Plant gage, mile 648.16.
5. At USGS gage, Highway Bridge, mile 429.7.
6. USGS gage at foot of Jefferson Street.
7. This is an emptying flow. The regulated flow was lower at the time of critical downstream flood conditions.
8. Highest daily average flow as corrected after official publication.

full use then can be made of the flood storage between the minimum and maximum headwater levels. Thus, even though the total storage volume filled from the beginning of the rise to the peak may be less under reservoir conditions than under natural conditions, the effective storage for peak reduction is greater with the reservoirs.

Tennessee River natural storage

As a check on flood routing computations under reservoir conditions, the routing method was first developed for and applied to natural pre-reservoir floods. This method, described in chapter 6, utilized inflows determined from stream gaging stations and rainfall and storage routing curves determined for reaches corresponding to the reservoirs. The family of curves in figure 79, page 124, shows the relation between inflow, outflow, and discharge, plus storage for the reach of the Tennessee River between Watts Bar and Chickamauga Dam sites. Similar curves were constructed for other reaches.

These curves were used to compute the natural flows in pre-reservoir floods for comparison with discharges at gaging stations, as at Chattanooga and Florence. A close agreement between computed and observed discharge hydrographs proved that, at least for natural conditions, the method was suitable. The conclusion was then drawn that the method was also suitable for reservoir conditions, and reservoir curves were constructed by a similar method.

Tributary reservoir inflow

In calculating the benefits from flood control, it is also necessary to determine the stage of the river before and after the construction of the storage reservoirs. Before the construction of the reservoirs the natural stages and discharges are determined from stream gages, while the regulated stages must be calculated.

On the other hand, after the dams are in operation all streamflows are regulated during floods. The natural flood stages, therefore, must be calculated, and the regulated stages are determined from observation of stream gages—hence the terms *pre-reservoir floods* and *post-reservoir floods.* If the streamflow records go back beyond the date of con-

TABLE 55.—*Peak discharges and stages below TVA dams and at other important locations, natural, observed and fixed-rule operation February 1950 flood.*[1]

	Computed natural		Observed[2]		Computed fixed rule[3]	
Location	Discharge, cubic feet per second	Stage, feet	Discharge, cubic feet per second	Stage, feet	Discharge, cubic feet per second	Stage, feet
Cherokee Dam	58,300	941.8	22,400[7]	931.8	22,000	931.2
Douglas Dam	39,200	882.1	21,700[7]	877.2	22,000	877.5
Knoxville[4]	103,800	819.1	58,600[7]	815.1	52,000	811.8
Fort Loudoun Dam	107,000	759.0	64,000	751.8	59,000	750.6
Fontana Dam	No flood	—	—	—	—	—
Norris Dam	76,500	844.5	26,300[7]	832.4	20,000[7]	830.7
Watts Bar Dam	248,000	702.6	164,000	693.0	110,000	688.3
Hiwassee Dam	No flood	—	—	—	—	—
Chickamauga Dam	254,000	663.8	179,000	652.9	142,000	650.0
Chattanooga	258,000	39.6	174,000[8]	28.4	143,000	25.8
Hales Bar Dam[5]	257,000	621.0	180,000	614.3	144,000	610.9
Guntersville Dam	250,000	576.9	192,000	572.8	193,000	571.9
Wheeler Dam	282,000	—	236,000	—	222,000	—
Wilson Dam	289,000	—	271,000	—	235,000	—
Florence	—	424.5	—	423.6	—	422.3
Pickwick Dam	300,000	392.1	268,000	389.7	242,000	388.2
Savannah	—	387.3	—	384.0	—	382.6
Kentucky Dam	334,000	342.8	291,000	342.4	314,000	341.4
Paducah[6]	—	339.8	—	339.3	—	338.6
Cairo	—	57.1	—	55.9	—	54.6

1. The TVA system was essentially completed at this time. Both the observed and fixed-rule operations of this flood were a continuation of the January 1950 flood causing the tributary reservoirs to be above normal level when this flood began.
2. Stages at tributary projects are the higher of twice-daily readings. At main-river dams they are from recorder charts. Reservoir discharges are the highest average 6-hour flows.
3. As regulated by the present-day system with the exception of South Holston, Watauga, and Boone Dams. Stages are from rating curves which represent average conditions.
4. At site of Old Water Plant gage, mile 648.16.
5. At USGS gage, Highway Bridge, mile 429.7.
6. USGS gage at foot of Jefferson Street.
7. This is an emptying flow. The regulated flow was lower at the time of critical downstream flood conditions.
8. A higher peak published by the USGS believed to be in error.

struction of a reservoir, as most of them do, the flood stages and discharges used will be partly natural and partly computed.

Pre-reservoir floods—For use in operation studies of pre-reservoir[1] floods, flood inflows into the tributary reservoirs were computed from reported discharges at gaging stations by correcting for differences in drainage areas. If no gaging station was in operation reasonably close to the reservoir site during the flood, inflows were computed from rainfall using a distribution graph based on other floods. The flood volume, or total runoff in this case, was determined from runoff records at other gaging stations. The availability of streamflow records on the Tennessee River at Knoxville, Chattanooga, Florence, and Johnsonville for most floods made it possible to estimate runoff volume where these data were not available on the tributaries.

The use of natural discharges for tributary reservoir inflow seems justified, because with those reservoirs there will be only minor differences between the natural discharge and what the inflow would have been if the reservoir had been constructed. This was shown by a comparison between the natural valley storage and total flood flow and reservoir storage. Because the volume occupied by a natural flood at any given time within the limits of a tributary reservoir is small compared with the flood volume, and consequently with the amount which may be stored, the natural valley storage lost by the construction and operation of tributary reservoirs is neglected in flood computations.

Estimated natural valley storage within the limits of seven of the tributary reservoirs and the storage reservation for flood control on January 1 are given in table 57. The ratio of the natural valley storage to the reservoir storage is greater in Douglas and Cherokee Reservoirs than in the other five reservoirs. This would be expected because of their relatively flatter slopes and wide flood plains.

With the relatively small volume of natural valley storage displaced by the reservoir, it seemed reasonable to assume that the difference between natural and reservoir inflow would be negligible.

Post-reservoir floods—In the case of post-reservoir floods when it is required to determine what the natural discharge would have been at each

1. It is realized that these pre-reservoir studies may strictly belong in chapter 7. However, it seemed more important that the continuity be preserved by including these floods with the post-reservoir floods.

TABLE 56.—*Peak discharges and stages below TVA dams and at other important locations, natural, observed, and fixed-rule operation January-February 1957 flood.*[1]

Location	Computed natural: Discharge, cubic feet per second	Computed natural: Stage, feet	Observed[2]: Discharge, cubic feet per second	Observed[2]: Stage, feet	Computed fixed rule[3]: Discharge, cubic feet per second	Computed fixed rule[3]: Stage, feet
South Holston Dam	28,000	—	8,900	—	8,000	—
Watauga Dam	11,500	—	Power flow only	—	4,000	—
Elizabethton	18,700	11.8	8,000	8.6	9,100	9.0
Kingsport[4]	52,000	14.0	15,800	6.7	15,800	6.7
Cherokee Dam	102,000	951.0	25,600[9]	932.7	27,000	932.9
Douglas Dam	110,000	874.3	33,500[9]	879.8	34,000	881.0
Knoxville[5]	205,000	833.3	67,500	816.6	69,750	817.3
Fort Loudoun Dam	212,000	776.1	82,600	757.7	72,000	755.9
Fontana Dam	61,500	—	8,400[9]	—	14,000[9]	—
Norris Dam	86,000	846.5	25,900[9]	832.3	18,000[9]	830.0
Clinton[6]	91,100	36.7	26,000[9]	19.9	19,600[9]	16.4
Watts Bar Dam	395,000	714.4	157,500	694.0	137,000	691.2
Hiwassee Dam	45,000	—	9,600[9]	—	10,000[9]	—
Chickamauga Dam	405,000	677.7	186,100	656.7	183,000	655.6
Chattanooga	412,000	54.0	206,000	32.2	190,000	31.0
Hales Bar Dam[7]	419,000	631.9	212,000[10]	618.1	198,000	616.0
Guntersville Dam	395,000	586.6	261,700	578.8	245,000	577.0
Wheeler Dam	430,000	—	349,500	—	346,000	—
Wilson Dam	441,000	—	372,000	—	373,000	—
Florence	—	432.0	—	427.3	—	428.3
Pickwick Dam	437,000	400.8	389,800	396.8	400,000	395.9
Savannah	—	396.7	—	392.4	—	393.1
Kentucky Dam	396,000	339.1	386,600	338.1	418,000	338.6
Paducah[8]	—	332.0	—	331.0	—	331.1
Cairo	—	47.2	—	45.7	—	46.0

1. In observed operation the tributary reservoirs were well below normal levels when the flood began. Fixed-rule operation started at normal levels.
2. Stages at tributary projects are the higher of twice-daily readings. At main-river dams they are from recorder charts. Reservoir discharges are the highest average 6-hour flows.
3. As regulated by present-day system. Stages are from rating curves which represent average conditions.
4. At gage below Highway 81 Bridge, mile 3.91.
5. At site of Old Water Plant gage, mile 648.16.
6. At site of discontinued gage, mile 59.0
7. USGS gage, Highway Bridge, mile 429.7.
8. USGS gage at foot of Jefferson Street.
9. This is an emptying flow. The regulated flow was lower at the time of critical downstream flood conditions.
10. Computed.

dam site, inflows were computed from storage changes in the reservoir and discharge of the outlet works. The assumption was made that this inflow would be nearly equal to natural discharge at that point, as was conversely assumed for the pre-reservoir floods. Changes in storage in each 6-hour period were combined with the actual discharge during the same period, adding the two amounts if the reservoir were rising and subtracting the storage increment from the actual discharge if the reservoir were falling. The discharge thus computed is the total amount during the 6-hour period, but for constructing a hydrograph it was assumed that this amount was a rate of flow occurring at the mid-point of the 6-hour period.

TABLE 57.—*Natural valley storage and tributary reservoir storage.*

Reservoir	Valley storage, acre-feet	Reservoir storage (January 1), acre-feet	Ratio of valley storage to reservoir storage
Cherokee	183,600	1,145,900	0.160:1
Douglas	227,000	1,311,200	0.173:1
Fontana	37,600	771,200	0.049:1
Norris	142,000	1,635,000	0,087:1
Chatuge	5,680	105,400	0.054:1
Nottely	7,720	110,000	0.070:1
Hiwassee	17,000	291,000	0.058:1

In both pre-reservoir and post-reservoir floods an allowance was made for the time of travel between the tributary dam or gaging station and the main-river reservoir or routing reach. These allowances were one-half day for Cherokee and Douglas and one day for Fontana, Norris, and Hiwassee.

Local inflow

Inflow into the main-river reaches or reservoirs is composed of the discharge at the next upstream dam, if any, and at any tributary gaging station or dam, combined with the inflow into the reaches or reservoirs from the areas not contributing to the discharges at the gaging stations or dams. This latter part is called local inflow, being composed principally of flow in the many small streams.

Pre-reservoir floods—For pre-reservoir floods, local inflow was computed in most cases from rainfall, using hydrograph distribution percentages determined from past floods. The total volume of the local inflow was computed from total flood volumes at main-river gaging stations and at tributary dam sites or gaging stations. Local inflow computations for all main-river floods were made on a daily basis. If shorter time intervals were required for a certain flood or part of a flood, values of local inflow were interpolated between the daily values.

Local inflows thus computed were added to the measured discharge at tributary gaging stations, or in the case of reservoir operation studies, with the discharge at the tributary dam, to obtain total inflow into the main-river reach or reservoir.

Post-reservoir floods—Local inflow for post-reservoir floods was computed by a method similar to that used for computing inflow into the tributary reservoirs. In this method the difference between discharges at the downstream dam and inflows from gaged tributaries or discharges from upstream dams is adjusted for the daily change in storage as represented by the change in headwater level on the flow-storage curves. Reported discharges and headwater elevations were entered on a form, and after assuming a trial local inflow to obtain the total inflow, the corresponding storage is read from the flow-storage curves. If the difference between inflow and outflow in the time interval is equal to the change in storage during that period, then the assumed trial local inflow was correct.

Many operation studies were made for the pre-reservoir floods by combining tributary reservoir releases with the local inflows and routing through the main-river reservoirs on the basis of some fixed rule or to obtain the ideal regulation. Local inflows were determined from recorded natural runoff or rainfall and used for reservoir studies. In the case of post-reservoir floods, however, the local inflow was computed entirely from regulated discharges and storage increments and was then used to compute natural discharges along the river and to make hypothetical reservoir operation studies. The results of these studies for the floods of 1946, 1950, and 1957 are given in tables 54, 55, and 56.

In these tables the difference between the "computed natural" data and the corresponding "computed fixed-rule" data is the amount of protection—in terms of reduced discharge and stage—it would be possible to give under the fixed-rule operation to areas that would have been flooded under natural conditions. A comparison of the "observed" and "computed fixed-rule" data in the three tables shows how nearly the fixed-rule operation was actually attained in the floods of 1946, 1950, and 1957.

Greatest reductions are obtained immediately below the tributary dams unless the tailwater is affected by backwater from another dam downstream, as in Hiwassee. Substantial reductions are also obtained at Knoxville and Chattanooga. The reduction becomes less, both in feet and in discharge, as the Ohio River is approached because of different physical characteristics of the river channel and because of the smaller portion of the area controlled by tributary storage reservoirs.

RESULTS OF ACTUAL OPERATION OF RESERVOIRS FOR FLOOD CONTROL

Volume stored in tributary reservoirs

In table 58 are listed the volumes of runoff which were stored in the ten tributary storage reservoirs above Chattanooga during ten flood periods. All ten reservoirs were not in operation until 1953. Volumes are for a period equal to that given for Chattanooga, but with a time of travel allowance from the reservoir to Chattanooga. The period during which regulated Chattanooga discharges were less than natural discharges determined the length of the selected period. The table shows that a substantial amount of reservoir inflow was stored, averaging nearly three-fourths during the critical periods in the ten floods. These stored volumes show that a high degree of control was accomplished.

Table 58 also shows that in each of the floods substantial amounts of empty storage space remained in the reservoirs for use in possible subsequent floods within the same flood season. This was especially desirable in the floods coming early in the flood season.

Crest reductions at Chattanooga and Cairo

Following the closure of Fontana Dam in 1944 and the completion of Kentucky Reservoir in 1945, floods occurred in three successive years (1946, 1947, 1948), affording an opportunity to test the effectiveness of the reservoir system and to gain experience in operation. The severe storm of January-February 1957 provided an even better test of the reservoir system. In January and February 1950 the highest flood since the record flood of 1937 occurred on the Ohio River at Cairo, Illinois, and was also a test of the TVA system. Tables 53, 54, 55, and 56 give natural and actual crest discharges and stages at each dam and at other points in the floods of 1937, 1946, 1950, and 1957. Figure 144 hydrographs, pages 256 through 260, show more completely the actual operation of the reservoir system in the 1950 flood.

The flood of 1957 would have been the second highest, and the 1946 flood would have been the fifth highest natural flood at Chattanoga if the reservoirs had not regulated the crest stages; and the floods of 1948 and 1946 would have been the fifth and eleventh highest, respectively, on the lower Tennessee River.

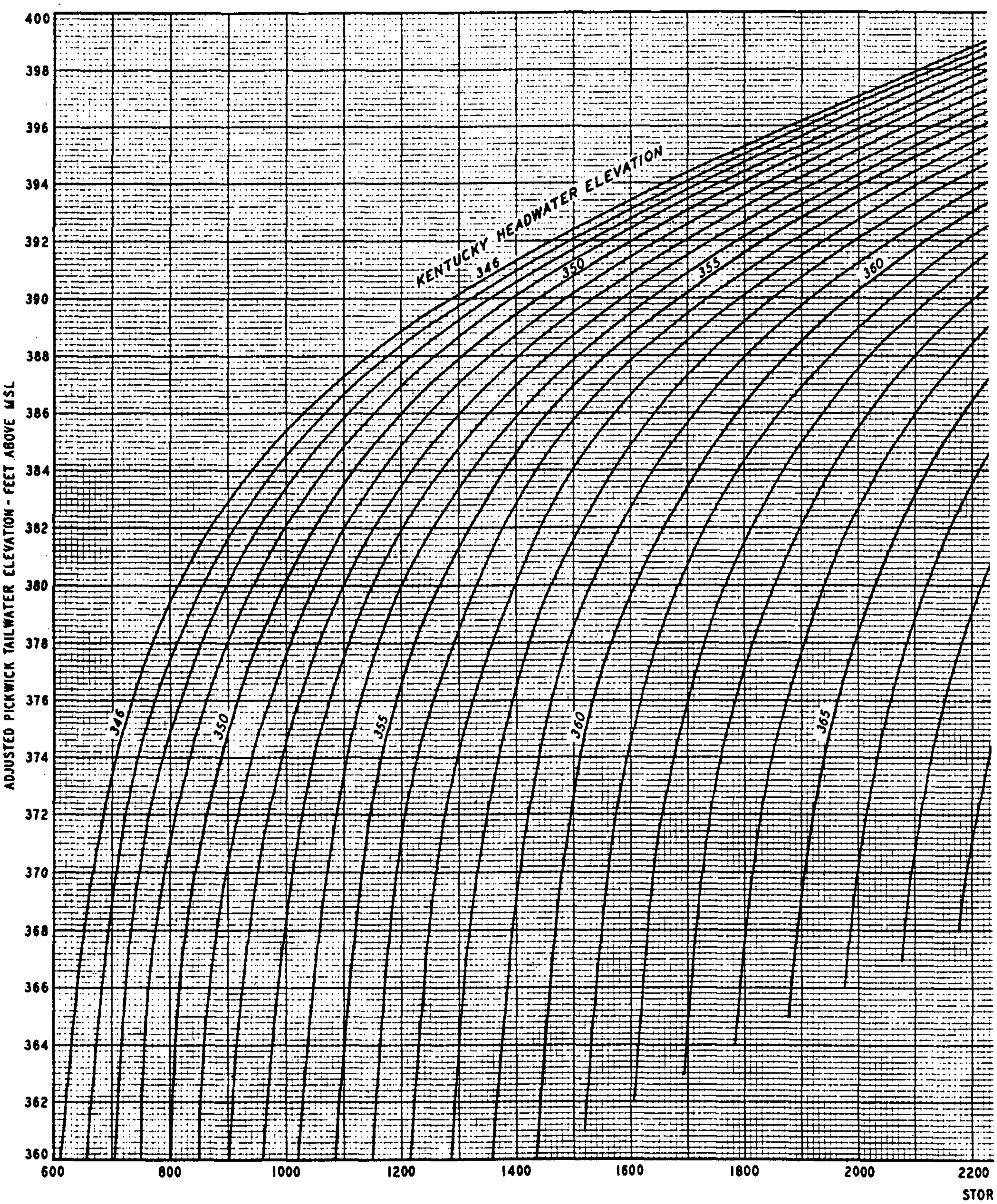
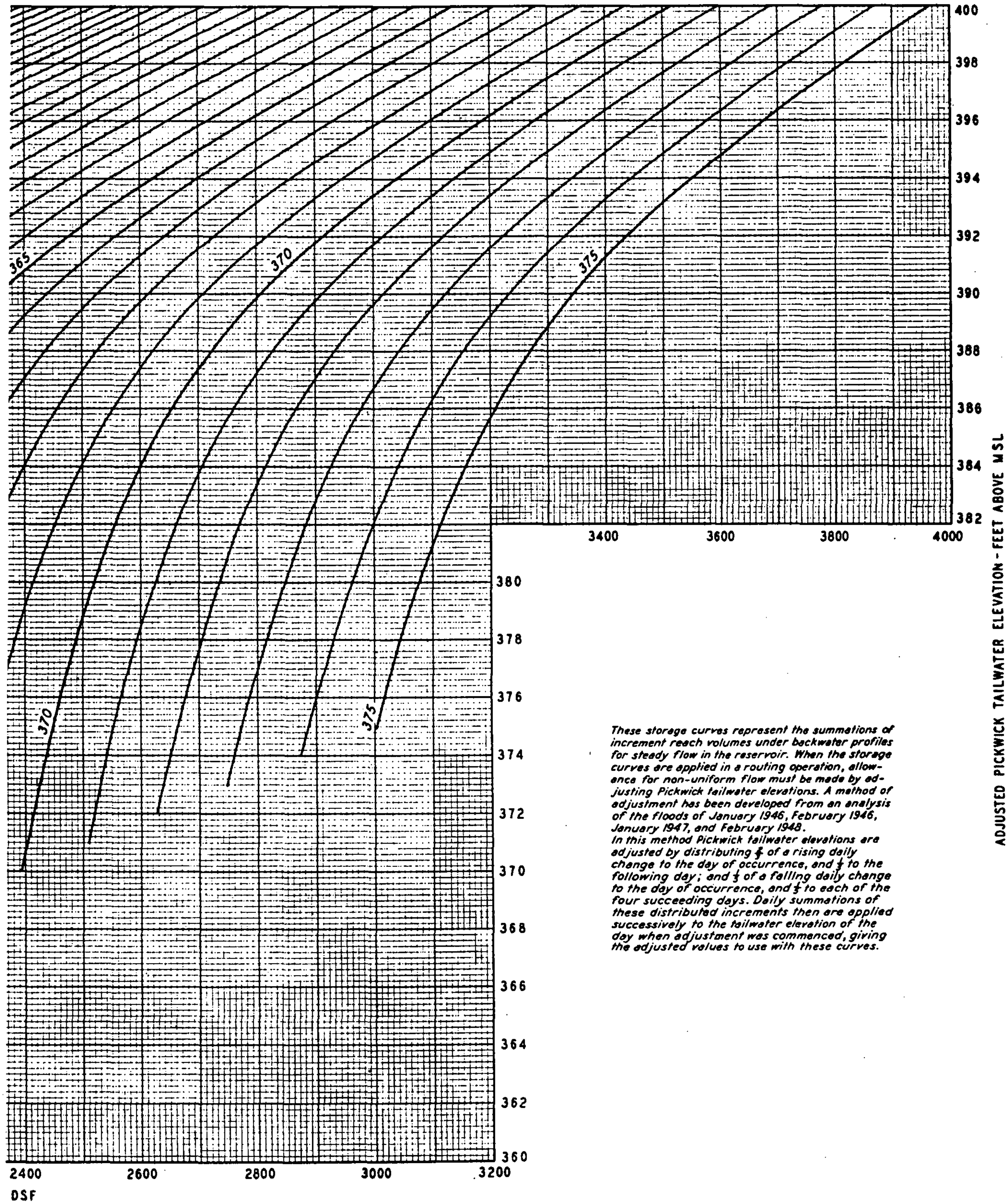


FIGURE 146.—*Routing c*



ntucky Reservoir.

TABLE 58.—*Flood volumes stored in tributary reservoirs during selected periods of flood reduction at Chattanooga.*

Chattanooga flood period	Reservoir	Inflow, 1000 day-second-feet	Storage: 1000 day-second-feet	Storage: Inches	Storage: Percent of inflow	Storage: Percent of total tributary storage	Remaining available storage, inches
3-24-36 to 4-15-36	Norris	495	495	6.32	100.0	100.0	8.79
12-27-42 to 1- 4-43	Cherokee	148	124	1.34	83.8	37.1	2.64
	Norris	184	151	1.93	82.1	45.2	7.03
	Nottely	13	13	2.26	100.0	3.9	4.36
	Chatuge	17	3	.59	17.6	0.9	.98
	Hiwassee	59	43	2.83	72.9	12.9	5.05
	Total	407[1]	334	1.70	82.1	100.0	4.58
3-27-44 to 4- 3-44	Cherokee	124	61	.66	49.2	22.6	1.14
	Douglas	166	102	.84	61.4	37.8	1.05
	Norris	131	74	.94	56.5	27.4	2.86
	Nottely	11	9	1.56	81.8	3.3	1.06
	Chatuge	13	6	1.18	46.2	2.2	1.69
	Hiwassee	50	18	1.18	36.0	6.7	0.91
	Total	485[1]	270	0.85	55.7	100.0	1.52
1- 8-46 to 1-14-46	Cherokee	178	128	1.39	71.9	21.4	4.86
	Douglas	267	196	1.60	73.4	32.7	3.80
	Fontana	125	57	1.35	45.6	9.5	7.49
	Norris	257	190	2.42	73.9	31.7	8.04
	Nottely	12	9	1.56	75.0	1.5	5.63
	Chatuge	11	9	1.77	81.8	1.5	7.63
	Hiwassee	46	10	0.66	21.7	1.7	8.43
	Total	892[1]	599	1.66	67.2	100.0	5.70
2-10-46 to 2-16-46	Cherokee	143	81	0.88	56.6	22.0	4.68
	Douglas	217	165	1.35	76.0	44.8	3.73
	Fontana	119	58	1.37	48.7	15.8	6.71
	Norris	152	47	0.60	30.9	12.8	8.98
	Nottely	18	5	0.87	27.8	1.4	6.24
	Chatuge	14	6	1.18	42.8	1.6	6.43
	Hiwassee	68	6	0.39	8.8	1.6	7.63
	Total	710[1]	368	1.02	51.8	100.0	5.84
1-17-47 to 1-26-47	Cherokee	280	227	2.46	81.1	24.9	4.96
	Douglas	325	248	2.03	76.3	27.3	3.10
	Fontana	181	143	3.38	79.0	15.7	7.33
	Norris	322	236	3.01	73.3	25.9	8.31
	Nottely	14	14	2.43	100.0	1.5	7.51
	Chatuge	14	14	2.75	100.0	1.5	9.16
	Hiwassee	70	29	1.91	41.4	3.2	7.36
	Total	1,206	911	2.52	75.5	100.0	5.54
2-13-48 to 2-19-48	Cherokee	177	153	1.66	86.4	22.9	5.66
	Douglas	198	153	1.25	77.3	22.9	3.81
	Fontana	95	78	1.84	82.1	11.7	10.35
	Norris	253	241	3.08	95.3	36.1	9.99
	Nottely	11	10	1.74	90.9	1.5	11.38
	Chatuge	11	11	2.16	100.0	1.6	14.57
	Hiwassee	33	22	1.45	66.7	3.3	9.51
	Total	777[1]	668	1.85	86.0	100.0	6.90
2- 1-50 to 2- 7-50	Watauga	18	15	1.19	83.3	2.8	8.82
	Cherokee	267	200	2.51	74.9	36.8	3.76
	Douglas	176	76	.62	43.2	14.0	4.11
	Fontana	56	—3	—.07	—5.4	—.6	6.68
	Norris	352	257	3.28	73.0	47.3	5.96
	Nottely	3	1	.17	33.3	.2	8.08
	Chatuge	4	2	.39	50.0	.4	8.08
	Hiwassee	20	—5	—.33	—25.0	—.9	8.23
	Total	890[1]	543	1.50	61.0	100.0	5.19

TABLE 58.—*Flood volumes stored in tributary reservoirs during selected periods of flood reduction at Chattanooga—Continued.*

Chattanooga flood period	Reservoir	Inflow, 1000 day-second-feet	Storage: 1000 day-second-feet	Storage: Inches	Storage: Percent of inflow	Storage: Percent of total tributary storage	Remaining available storage, inches
1-21-54 to 1-28-54	Watauga	22	11	.87	50.0	1.7	19.87
	South Holston	32	22	1.16	68.8	3.5	14.50
	Boone	48	18	1.00	37.5	2.9	1.31
	Cherokee	107	93	2.18	86.9	14.8	14.10
	Douglas	276	209	1.71	75.7	33.2	3.73
	Fontana	126	106	2.51	84.1	16.9	9.60
	Norris	127	123	1.57	96.8	19.6	12.27
	Nottely	15	15	2.60	100.0	2.4	11.22
	Chatuge	16	16	3.15	100.0	2.5	7.44
	Hiwassee	53	16	1.05	30.2	2.5	6.67
	Total	771[1]	629	1.74	81.6	100.0	8.80
1-28-57 to 2- 9-57	Watauga	56	52	4.13	92.9	3.8	10.75
	South Holston	16	94	4.97	81.0	6.9	5.86
	Boone	109	19	1.06	17.4	1.4	1.56
	Cherokee	368	233	5.45	63.3	17.1	10.52
	Douglas	504	338	2.77	67.1	24.8	2.92
	Fontana	231	171	4.05	74.0	12.5	7.29
	Norris	499	371	4.74	74.3	27.2	6.86
	Nottely	18	18	3.13	100.0	1.3	8.55
	Chatuge	20	20	3.93	100.0	1.5	9.75
	Hiwassee	102	47	3.09	46.1	3.5	6.97
	Total	1,907[1]	1,363	3.78	71.5	100.0	5.90

1. Because inflows into certain reservoirs include releases from upstream projects, the sum of the individual reservoir inflows has been adjusted to represent total system tributary inflow.

The 1950 crest stage actually was the third highest known at Cairo and about the tenth highest in terms of discharge. Without the TVA reservoirs it would have been the sixth highest in terms of discharge.

The difference between the actual and natural hydrographs of figure 144 is the effect of the operation of the reservoir system. Some of the crest reduction on the Tennessee River is the result of storage in the tributary reservoirs, some is possible because of acceleration of the rising limb of the hydrograph and later storage in the main-river reservoirs, and at some points there is stage reduction because of improved channel carrying capacity.

Table 59 shows the computed natural and actual regulated crest stages at Chattanooga for floods that have occurred since the TVA reservoirs were put in operation. Areas flooded within the city limits for each condition are also given.

Actual and computed natural crest stages at Cairo and the stage reductions obtained since the closure of Kentucky Dam are given in table 60. The flood crest height at that city is the criterion of damage on the lower Ohio and Mississippi Rivers, and flood stage reductions at Cairo, therefore, are the objective of the regulation of Tennessee River releases at Kentucky Dam. Up to the present time a full flood control operation of Kentucky Reservoir to its maximum elevation (375) has not been required. The highest flood at Cairo since 1937 occurred in 1950 when the crest stage was 55.3 feet on January 19 and 55.9 on February 15. Natural crests would have been 57.2 feet and 57.1 feet, respectively. Kentucky Reservoir was filled to elevation 368.81 on January 24 in regulating the Cairo crest.

Table 60 lists only major floods since closure of Kentucky Dam. In those floods for which estimates of natural crest stage were not made, little or no regulation by Tennessee River reservoirs was possible. For example, the floods of May 2, and June 19, 1947, reached crest stages of 45.8 and 45.2 feet, respectively, but because of the extremely low flow in the Tennessee River, no flood control operation of Kentucky Reservoir was practicable. Careful attention was required, however, to ensure that the flood crest would not be raised.

Because all reservoir adjustments were not completed in March and April 1945, only limited use could be made of Kentucky storage during those floods. Nevertheless, system operations resulted in a crest reduction from 55.4 feet to 53.9 feet.

Other flood crests occurred before completion of Kentucky Dam which were regulated by the reservoirs in the Tennessee River Basin. The floods of April 1936 and February 1937 reached stages of 52.7 and 59.5 feet respectively, and it has been estimated they would have been from 3 to 6 inches higher had it not been for storage in Norris Reservoir on the Clinch River.

TABLE 59.—*Flood crest stages and areas flooded at Chattanooga—1936-1958.*[1]

Date	Projects completed above Chattanooga		Stage in feet			Acres flooded in city	
	Tributary	Main-river	Actual	Computed natural	Reduction	Actual flood	Natural flood
March 29, 1936	1	0	37.1	41.3	4.2	3300	4600
April 9, 1936	1	0	35.4	38.8	3.4	2700	3700
January 4, 1937	1	0	33.0	35.5	2.5	2200	2800
December 30, 1942	5	2	35.8	39.7	3.9	2800	4000
February 19, 1944	6	3	23.0	34.6	11.6	700	2500
March 30, 1944	6	3	31.7	37.8	6.1	1800	3400
January 9, 1946	7	3	35.7	45.8	10.1	2800	6300
February 11, 1946	7	3	29.7	36.8	7.1	1600	3100
January 21, 1947	7	3	31.9	44.5	12.6	1900	5900
February 14, 1948	7	3	33.8	44.3	10.5	2400	5800
November 29, 1948	7	3	29.0	35.7	6.7	1500	2800
January 6, 1949	8	3	29.5	36.3	6.8	1600	3000
February 2, 1950	8	3	28.4	39.6	11.2	1400	4000
March 15, 1950	8	3	26.7	32.5	5.8	1100	2100
March 30, 1951	9	3	25.8	35.6	9.8	1000	2800
January 22, 1954	10	3	29.8	42.0	12.2	1600	4900
March 23, 1955	10	3	22.5	35.8	13.3	600	2800
February 4, 1956	10	3	27.4	32.2	4.8	1200	2000
April 17, 1956	10	3	17.8	34.0	16.2	300	2400
February 2, 1957	10	3	32.2	54.0	21.8	2000	8300
April 5, 1957	10	3	14.9	30.9	16.0	160	1800
November 20, 1957	10	3	29.6	36.8	7.2	1600	3100
April 30, 1958	10	3	23.3	30.6	7.3	700	1700
May 10, 1958	10	3	18.5	31.1	12.6	300	1800

1. The system reduced nine additional floods which would have exceeded 30 feet by amounts that would not have resulted in significant flood damage.

Comparison of actual operation with planning studies

The curves shown in figure 147 show regulated peak discharges along the Tennessee River in terms of percent of natural peak discharge. This chart is a convenient means of representing the effect of regulation and also of comparing the peak reductions actually obtained with those which might have been obtained if another type of operation had been followed. In figure 147 hypothetical fixed-rule operations in the floods of 1861 and 1948 are compared with actual operation in the 1948 flood and with reduction in proportion to controlled tributary drainage area.

The drainage area curve in figure 147 shows for any point along the river the uncontrolled area below the major tributary storage reservoirs as a percent of the total area above that point. For example, at Chattanooga (mile 464) the uncontrolled area is 37 percent of the total area. This curve, which represents a kind of limit in the regulation of flood crests, shows the greatest reduction in peak discharge which could be obtained with complete storage in the tributary reservoirs during a hypothetical flood crest in which all parts of the Basin contributed equally.

In the actual operation for the 1948 flood, the main-river reservoirs were only partially filled, but the entire tributary inflow was stored except for the nominal release for power generation. The curve for this flood is similar to the drainage area curve, running parallel to it but about 30 to 40 percent higher. The increase above natural peak discharge at Kentucky is the result of drawing down that reservoir at the time of high inflows, but this did not mean an increase in peak stage below Kentucky Dam. The areas of high rainfall and runoff on the Emory River Basin in eastern Tennessee and near Florence, Alabama, produced irregularities in the curve at Watts Bar Dam and at Wilson Dam, respectively.

Effect of reservoir groups on Chattanooga stage

The effect at Chattanooga of tributary or main-river reservoirs considered as groups was determined for the 1946, 1947, 1948, 1950, 1954, and 1957 floods by assuming that the main-river reservoirs were not built and that the actual tributary reservoir operation would be repeated. Inflows were routed downstream through the three reaches of the main river with natural routing curves, and hydrographs of discharge at Chattanooga were obtained. The effect of the tributary reservoirs is the difference between the hydrographs thus computed and the natural hydrographs, and the effect of the main-river reservoirs

TABLE 60.—*Flood crest stages at Cairo—1945-1960.*

Flood	Actual stage, feet	Stage without Tennessee River regulation, feet	Reduction, feet
Mid-March 1945	53.9	55.4	1.5
Late-March 1945	53.9	54.3	0.4
April 1945	53.7	54.3	0.6
May 1945	44.3	(1)	(1)
June 1945	44.7	(1)	(1)
January 1946	52.1	53.5	1.4
February 1946	45.9	46.3	0.4
January 1947	41.0	42.9	1.9
April 1947	47.1	48.0	0.9
May 1947	45.8	(1)	(1)
June 1947	45.1	(1)	(1)
February 1948	46.8	48.7	1.9
March 1948	45.6	46.1	0.5
Early-April 1948	51.6	53.4	1.8
Late-April 1948	47.9	49.0	1.1
January 1949	50.7	51.3	0.6
February 1949	49.3	50.5	1.2
April 1949	46.8	46.9	0.1
January 1950	55.3	57.2	1.9
February 1950	55.9	57.1	1.2
April 1950	46.8	47.3	0.5
February 1951	49.0	49.0	0
Mid-April 1951	47.5	47.5	0
Late-April 1951	46.2	46.7	0.5
February 1952	47.7	(1)	(1)
March 1952	50.7	51.2	0.5
May 1952	44.0	(1)	(1)
March 1955	50.1	50.9	0.8
February 1956	40.1	41.0	0.9
February 1956	43.7	45.8	2.1
March 1956	40.7	42.3	1.6
February 1957	45.7	47.2	1.5
April 1957	43.8	46.8	3.0
May 1958	43.1	46.2	3.1
July 1958	43.8	44.8	1.0
February 1959	38.3	40.5	2.2
February 1959	40.3	41.6	1.3
April 1960	47.4	50.1	2.7

1. Detailed estimates not available.

is the difference between those hydrographs and the actual.

Table 61 gives peak discharges at Chattanooga and stages on the Chattanooga gage for various operating combinations of the tributary and main-river reservoirs. It shows that the tributary reservoirs as a group caused reductions ranging from a minimum of 6.8 feet to a maximum of 16.1 feet. Similarly, the main-river reservoirs as a group caused additional reductions ranging from 0.1 to 5.7 feet. In all six floods, mainstream reservoir fixed-rule operation gave lower stages than other operations.

Similar studies of the effects of groups of reservoirs on Cairo stages gave the results shown in table 62. In one of these studies, "Reservoirs down to Hales Bar," it was assumed that the actual operation of all reservoirs above Hales Bar would be repeated and that there were no dams below that point. The difference between Cairo stages thus computed and natural Cairo stages is the effect of the reservoirs above Hales Bar. In another of these studies, "Reservoirs down to Pickwick," it was assumed that the actual operation of all reservoirs above Pickwick would be repeated and that Kentucky Dam was not built. The difference between Cairo stages on this assumption and those on the assumption with reservoirs above Hales Bar is the effect of Guntersville, Wheeler, Wilson, and Pickwick Reservoirs. The difference between Cairo stages on the assumption of all reservoirs down to Pickwick and the actual (all reservoirs) stages is the effect of Kentucky Reservoir. Table 62 lists Kentucky discharges and tailwater elevations, Paducah elevations, and Cairo stages for five floods occurring since the completion of the Kentucky project, computed on the basis of the above assumptions.

Table 62 indicates that in all but the 1957 flood Kentucky Reservoir was the largest contributor to the Cairo stage reduction, as would be expected because of its location. In the 1957 flood, Cairo stages

TABLE 61.—*Comparison of operations for Chattanooga.*

Tributary reservoir operation	Main-river reservoir operation	Maximum discharge, cubic feet per second	Chattanooga stage, feet	Reduction by each succeeding operating combination, feet
		1946 flood		
Natural	Natural	320,000	45.8	—
Actual	Natural	244,000	38.1	7.7
Actual	Actual	222,000	35.7	2.4
Actual	Fixed rule	180,000	29.9[1]	5.8
		1947 flood		
Natural	Natural	307,000	44.5	—
Actual	Natural	202,000	33.4	11.1
Actual	Actual	188,000	31.8	1.6
Actual	Fixed rule	173,000	29.0[1]	2.8
		1948 flood		
Natural	Natural	305,000	44.3	—
Actual	Natural	239,000	37.5	6.8
Actual	Actual	205,000	33.8	3.7
Actual	Fixed rule	187,000	30.6[1]	3.2
		1950 flood		
Natural	Natural	258,000	39.6	—
Actual	Natural	161,000	28.5	11.1
Atcual	Actual	174,000	28.4[2]	0.1
Actual	Fixed rule	148,000	26.3[1]	2.1
		1954 flood		
Natural	Natural	276,000	41.4	—
Actual	Natural	212,000	34.6	6.8
Actual	Actual	183,000	29.8[2]	4.8
Actual	Fixed rule	174,000	29.1[1]	0.7
		1957 flood		
Natural	Natural	412,000	54.0	—
Actual	Natural	242,000	37.9	16.1
Actual	Actual	206,000	32.2[2]	5.7
Actual	Fixed rule	178,000	29.5[1]	2.7

1. For Hales Bar spillway conditions since the spring of 1948.
2. Not comparable with floods prior to spring of 1948 due to changes in Hales Bar spillway.

TABLE 62.—*Comparison of operations for Cairo.*

Reservoir system	Peak discharge or stage: Kentucky discharge, cubic feet per second	Kentucky tailwater elevation, feet	Paducah elevation, feet	Cairo stage, feet	Reduction[1] by each succeeding operating combination, feet
		January 1946 flood			
Natural	357,000	338.6	334.5	53.5	—
Reservoirs down to Hales Bar	333,000	337.5	333.8	53.2	0.3
Reservoirs down to Pickwick	356,000	337.6	333.6	53.1	0.1
Actual (all reservoirs)	400,000	337.3	332.2	52.1	1.0
		February 1948 flood			
Natural	378,000	339.7	334.0	48.7	—
Reservoirs down to Hales Bar	357,000	338.0	332.8	47.8	0.9
Reservoirs down to Pickwick	372,000	337.6	332.4	47.7	0.1
Actual (all reservoirs)	444,000	338.9	332.0	46.7	1.0
		January 1950 flood			
Natural	241,000	343.0	341.1	57.2	—
Reservoirs down to Hales Bar	249,000	342.6	341.4	57.1	0.1
Reservoirs down to Pickwick	252,000	342.3	341.2	57.2	(0.1)
Actual (all reservoirs)	375,000	342.6	338.6	55.3	1.9
		February 1950 flood			
Natural	334,000	344.3	341.3	57.1	—
Reservoirs down to Hales Bar	308,000	343.1	340.7	56.6	0.5
Reservoirs down to Pickwick	307,000	343.1	340.8	56.7	(0.1)
Actual (all reservoirs)	291,000	342.4	339.9	55.9	0.8
		1957 flood			
Natural	396,000	339.1	332.2	47.2	—
Reservoirs down to Hales Bar	331,000	335.3	330.2	45.3	1.9
Reservoirs down to Pickwick	361,000	336.5	330.6	45.4	(0.1)
Actual (all reservoirs)	386,600	338.1	331.0	45.7	(0.3)

1. Figures in parentheses denote increase.

TABLE 63.—*Effect of Watauga and South Holston Reservoirs in Kingsport maximum probable flood.*

Assumed operation		Watauga		South Holston		Kingsport	
Sluice opening	Initial reservoir elevation	Peak discharge, cubic feet per second	Maximum reservoir elevation[1], feet	Peak discharge, cubic feet per second	Maximum reservoir elevation[1], feet	Peak discharge, cubic feet per second	Stage[1,2] feet
Natural	—	72,000	—	108,000	—	220,000	30.6
0	Normal March 15	10,000	1979.2	18,000	1747.0	138,000	23.5
0	Maximum normal	26,000	1982.1	60,000	1751.6	138,000	23.5
0	Spillway crest	52,000	1986.5	85,000	1753.8	138,000	23.5
½	Normal March 15	9,000	1976.6	15,000	1745.5	148,000	24.4
½[3]	Maximum normal[3]	23,000	1980.5	56,000	1750.8	148,000	24.4
½	Spillway crest	52,000	1985.6	85,000	1753.2	150,000	24.5
Full	Normal March 15	12,000	1973.1	11,000	1743.8	161,000	25.5
Full	Maximum normal	20,000	1978.7	55,000	1750.0	161,000	25.5
Full	Spillway crest	52,000	1984.5	84,000	1752.8	161,000	25.5
Variable[4]	Maximum normal	23,000	1979.8	59,000	1750.3	140,000	23.7

1. Showing computed stages to tenths of a foot does not imply such accuracy. It is shown here only for the purpose of ready identification of the original calculations.
2. Gage at mile 3.71.
3. Used for design.
4. In accordance with a fixed-rule operating guide.

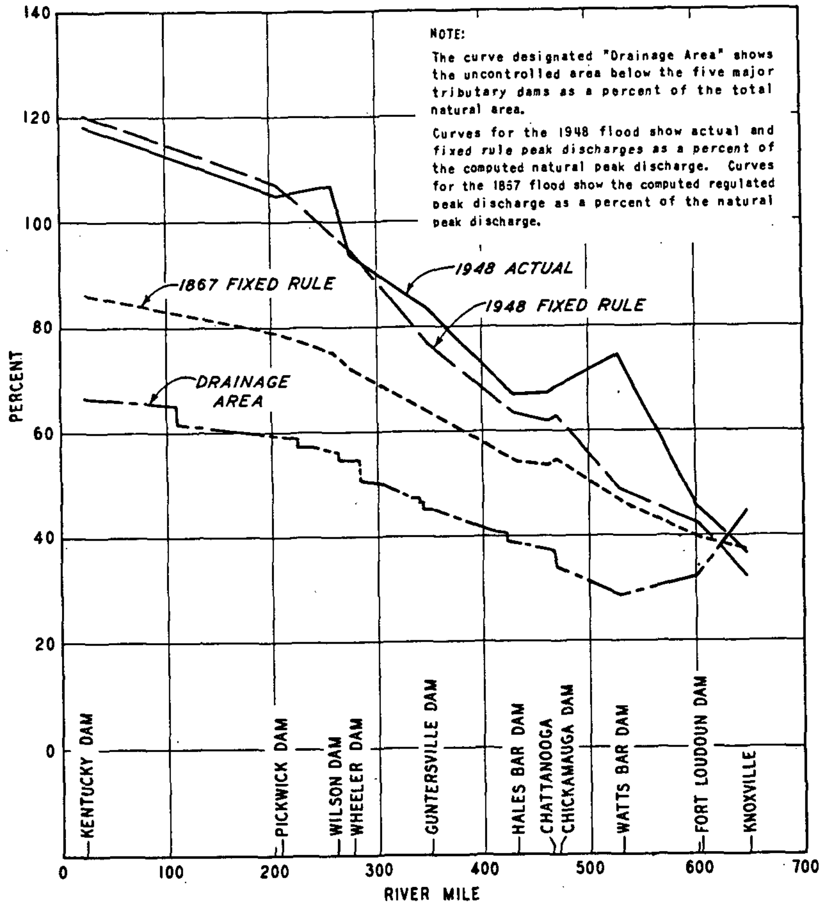


FIGURE 147.—*Relative flood crest reductions—Tennessee River—1867 and 1948.*

were not high enough to require any storage by Kentucky. In the four other floods, Kentucky's contribution could have been much larger if a full-scale operation of that reservoir had been required. In the 1950 and 1957 floods small increases resulted from the Guntersville, Wheeler, Wilson, and Pickwick operations, but again full use of the storage was not made in those reservoirs. In the 1957 flood, operation of those reservoirs plus Kentucky caused a slightly larger increase in Cairo crest stage.

Operation for Kingsport and Elizabethton

Flood storage in Watauga Reservoir on the Watauga River was planned primarily for the relief of flooding at Elizabethton 11 miles downstream, and at Kingsport 51 miles downstream. Flood storage in South Holston Reservoir on the South Fork Holston River was planned primarily for flood relief at Kingsport 44 miles downstream. In addition, a small flood storage reservation is provided in Boone Reservoir, located on the South Fork about 10 miles upstream from Kingsport.

The greatest flood crest reduction at Elizabethton and Kingsport would be attained by storing all inflow to South Holston and Watauga Reservoirs until the crest discharge from the local area between these dams and Kingsport has passed. Such an operation would be possible, except in the case of extreme floods, because of the relatively large storage reservation available compared to expected flood volumes. However, since the magnitude of a flood will not be known in advance, a safer operation is to discharge turbine flow at the beginning of the flood and then, as the inflow increases, release increasing rates of flow through the sluices. This type of operation is similar to that of detention basins where the outflow is through an uncontrolled outlet. In every flood that exceeds turbine capacity, storage of flood water will take place. This will produce a positive crest reduction downstream provided, of course, that the stored water would have otherwise contributed to the downstream peak.

Table 63 gives peak discharges and reservoir elevations in Watauga and South Holston Reservoirs, and discharges and stages at Kingsport for the Kingsport maximum probable flood under several assumptions of sluice opening and initial reservoir stage. Hydrographs in figure 148 show natural and computed regulated discharges for this flood and for two large actual floods—August 1940 and May 1901. Since flood stage at Kingsport corresponds to a flow of about 45,000 cubic feet per second, it is obvious from a study of the hydrographs that Watauga and South Holston could reduce the largest known floods to below a damaging stage at Kingsport. In order to provide Kingsport with complete protection against the maximum probable flood, however, additional storage or local protection works must be provided. Hydrographs in figure 149 show the effect of flood regulation at Elizabethton.

Boone Reservoir is operated during periods of high flow for flood regulation downstream, especially at Kingsport. Outflows from Fort Patrick Henry Reservoir where no flood storage is provided parallel those at Boone Reservoir — about 10 miles upstream — with allowances made for the time of water travel and local inflow between the two projects. Operation of these two projects is unique for several reasons. There is a rapid concentration of flood runoff from the local area, necessitating alert and rapid operation to reduce the crest flow. There is a relatively small volume of flood storage space

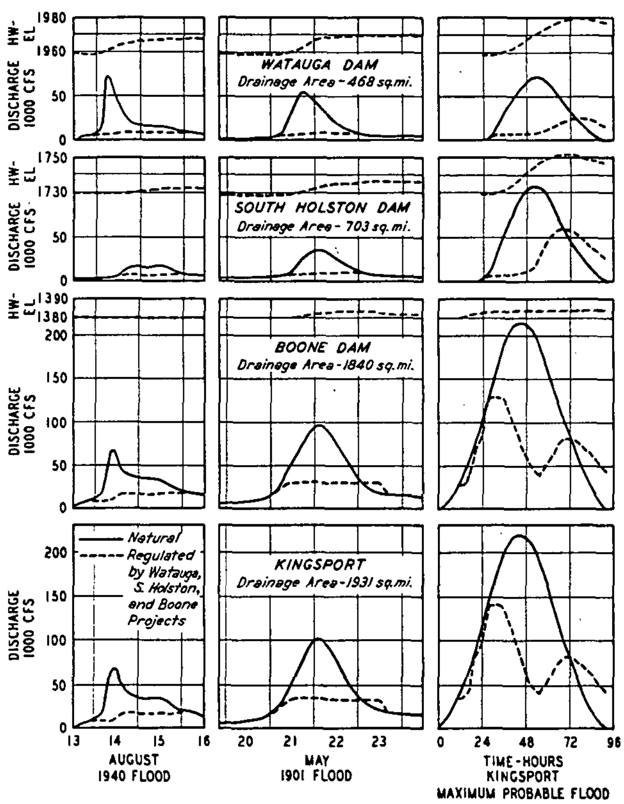


FIGURE 148.—*Effect of flood regulation at Kingsport, Tennessee.*

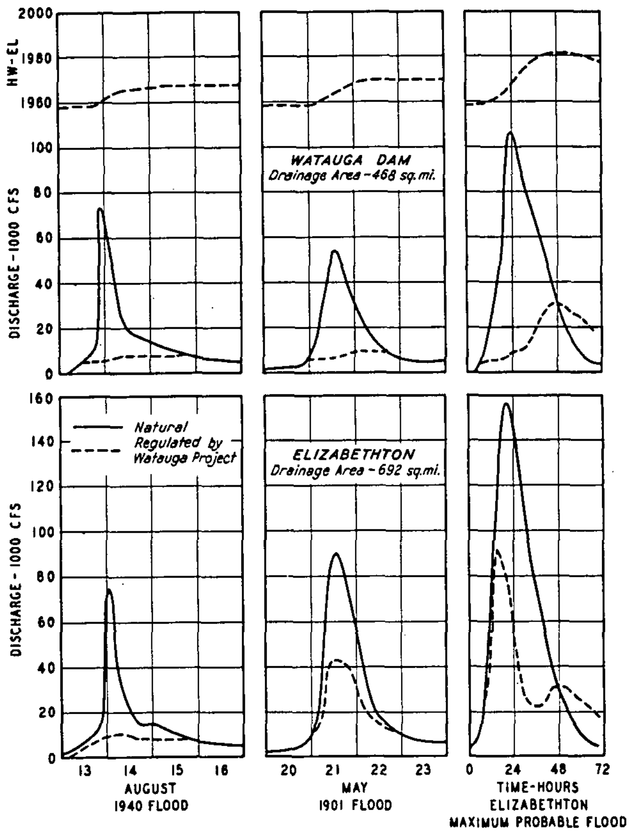


FIGURE 149.—*Effect of flood regulation at Elizabethton, Tennessee.*

available, all of it being in Boone Reservoir. The location of the two projects is such that an interruption of communications with Knoxville is possible. These circumstances made it imperative to be able to regulate flood discharges locally at the two dams rather than from instructions issued at Knoxville. Fixed rules were prepared, therefore, for the operation of Boone and Fort Patrick Henry Reservoirs so that operation for flood reduction could be handled by personnel stationed at or near those two dams.

Table 64 lists the actual stage at Kingsport and Elizabethton and the computed natural stage for floods occurring since the completion of Watauga, South Holston, Boone, and Fort Patrick Henry Reservoirs. None of these floods would have produced serious damage even without regulation, but the reductions in crest stages show the degree of control possible with the reservoirs.

Operation of proposed detention basins on French Broad River above Asheville

The system of seven detention-type reservoirs proposed for the control of floods at Asheville, North Carolina, and at agricultural lands lying along the

TABLE 64.—*Flood crest stages at Kingsport and Elizabethton 1949-1957.*

Year and Date	Kingsport[1] Observed crest stage, feet	Kingsport[1] Computed natural[3] crest stage, feet	Kingsport[1] Reduction, feet	Elizabethton[2] Observed crest stage, feet	Elizabethton[2] Computed natural[3] crest stage, feet	Elizabethton[2] Reduction, feet
1949						
March 19	6.0	7.1	1.1	—	—	—
1950						
January 31	8.6	9.3	0.7	—	—	—
February 9	6.7	7.3	0.6	—	—	—
May 12	7.3	7.7	0.4	—	—	—
December 7	5.3	8.9	3.6	9.0	13.2	4.2
1951						
December 21	4.8	7.0	2.2	7.4	9.0	1.6
1953						
February 21	3.1	8.8	5.7	6.8	10.2	3.4
1954						
January 16	3.4	7.7	4.3	—	—	—
January 22	3.5	12.6	8.7	9.5	12.7	3.2
1955						
February 6	—	—	—	6.3	9.9	3.6
March 16	—	—	—	7.4	11.0	3.6
March 17	4.5	10.9	6.4	—	—	—
March 18	7.2	12.8	5.6	8.1	11.0	2.9
April 14	—	—	—	5.4	11.2	5.8
1956						
February 17	—	—	—	7.8	9.3	1.5
February 18	4.8	7.6	2.8	—	—	—
April 16	6.9	13.6	6.7	9.1	13.2	4.1
1957						
January 29	—	—	—	8.6	11.8	3.2
February 1, 2	6.6	14.0	7.4	—	—	—
February 16	6.7	10.6	3.9	7.6	9.8	2.2
April 5	4.6	12.3	7.7	9.1	14.0	4.9
April 8	5.0	10.7	5.7	8.2	10.4	2.2

1. Floods since closure of Watauga Dam, December 1, 1948, exceeding a natural crest stage of 7.0 feet on the gage at mile 3.71. Flood stage equals 12 feet.
2. Floods since December 1,1948, exceeding a natural crest stage of 9.0 feet. Flood stage equals 14.0 feet.
3. Showing computed stages to tenths of a foot does not imply such accuracy. It is shown here only for the purpose of ready identification of the original calculation.

French Broad River above Asheville would be automatic in its operation, requiring no full-time attendants at the dams. The outlet works at each dam would consist of a sluice opening through the dam, without a regulating gate, and a free overfall spillway at a higher level for preventing overtopping of the dam. The capacity of the sluice opening would be such that the channel carrying capacity below the dam would not be exceeded except when the spillway comes into use in extreme floods. This sluice capacity is 1 inch of runoff per day from the drainage area at Swannanoa River, Cane Creek, and Clear Creek Dams, three-fourths of an inch per day at Little River Dam, and one-half inch per day at Mills and Davidson River Dams and at the Brevard Dam on the French Broad River.

Because of the small size of the areas above the dams and the intensity of the flood-producing storms, floods rise rapidly, sometimes from low stage to flood crest in less than 12 hours. It would not be economical to have personnel in full-time attendance at each of these dams to operate gate-controlled outlets. Also, with gated conduits a competent hydrologic force would need to be stationed nearby, and many more gages would be required to predict the inflows on various streams. As proposed, the operation would be automatic with open conduits.

Possible reductions in several actual floods and in the maximum probable flood at Asheville by the proposed system of seven detention basins in the upper French Broad Basin are shown in figure 78, page 122. Here, the maximum known flood cannot be reduced by reservoirs below the damaging stage, and levees would have to be constructed to give complete protection against this flood and one as great as the maximum probable flood.

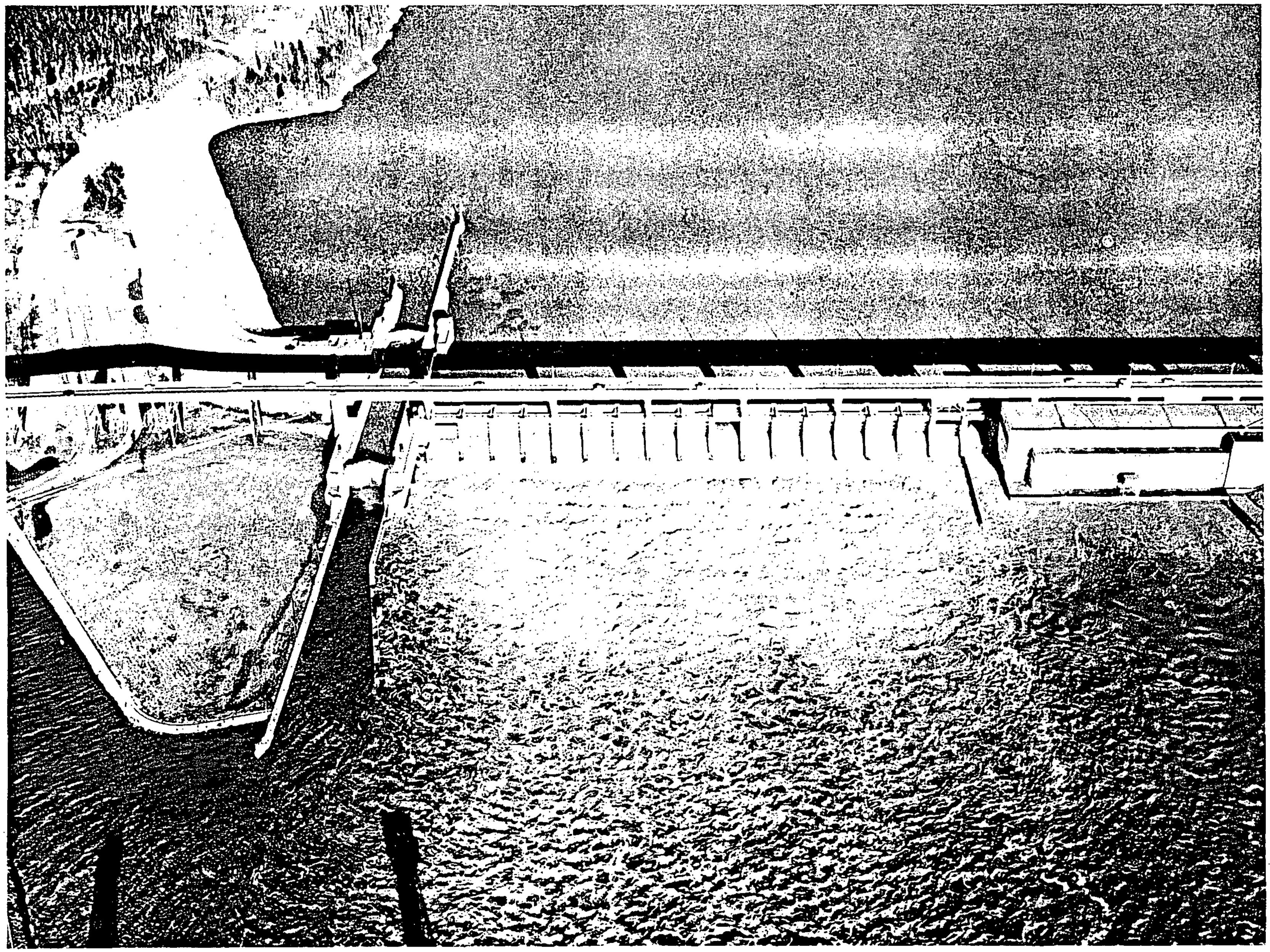


FIGURE 150.—*Chickmauga Dam, just upstream from Chattanooga, regulating flood waters for damage prevention at that city.*

APPENDIX C

FLOOD DAMAGE APPRAISAL CHATTANOOGA, TENNESSEE--1938

This appendix includes that part of the report, "The Chattanooga Flood Control Problem," (House Document No. 91, 76th Congress, 1st Session, 1939) which describes the methods used in making the 1938 flood damage appraisal of Chattanooga. This appraisal is the basis for the determination—as discussed in chapter 12—of potential flood damage in that city.

The methods used in appraising various types of flood damage are first discussed, the appraisal data are then applied to past floods and to the design flood, and the concluding discussion of this appendix covers intangible flood damage.

APPRAISAL OF FLOOD DAMAGES

The expenditure for flood control which may be justified depends both on the amount of flood damage which could be prevented and on the benefits resulting. Certain of these benefits are the direct result of prevention of damages; others arise from resulting new development within the flooded area, with increases in value and population that otherwise would not occur.

Because Chattanooga has not experienced since 1886 a flood large enough to invade the principal business district, no data are available on the actual damage by a major flood to the city as it is today. Therefore, an appraisal of flood damage to the present city was made, including the area on both banks of the Tennessee River and the adjacent community of Rossville, Ga., but excluding that portion of Chattanooga east of Missionary Ridge.

The purpose of the appraisal was to determine (1) the flood damage to the present city of Chattanooga if the highest recorded river stage were repeated, and (2) the damage which would now be caused by lesser floods, so that a calculation could be made of the average annual flood damage which would be caused by a repetition of the floods from 1867 to 1938 during a period of the same length and with improvements as they are at present.

Maps were prepared showing the portion of the city which would be flooded by the highest stage of record, that of 1867, and by a flood 10 feet lower, approximately that of 1917, neither of which has since been equaled. Elevations at practically all street intersections within the 1867 flood area were obtained. An appraisal form for each residential block, each commercial block, and each industry was prepared showing block number, street names, surface elevation, and 1867 high-water elevation. Appraisers then examined each block. In the residential areas, the houses were counted, the number of rooms in each house estimated, the character of construction and the number of floors noted, and the houses classified into one of three classes according to value of house and contents. Commercial establishments of each kind were counted, size of building and basement was noted, and each business divided into two classes according to apparent value of fixtures and inventory. Average inventories of fixtures and stocks were obtained from the merchants for each kind of business—for the best, the average, and the poorest. Each industry was visited, and, with the aid of a representative of the industry, an appraisal was made of the damage to buildings, machinery, and stocks which would be caused by an 1867 flood stage. Where possible, the average number of people employed by each industry was obtained.

Residential damage—The flood damage caused to residential property was computed on a room basis. The unit rates used were based in part on the results obtained by the Tennessee Valley Authority from a house-to-house survey of the actual damage caused to all such property by the 1937 flood at Paducah, Ky. That flood inundated 6,900 houses to a depth of from 4 to 10 feet above the ground floor—most of them to a depth of about 6 feet. These results were supplemented by studies of the cost of repairing, refinishing, and refurnishing houses of the best, medium, and poorest types.

Consideration also was given to the depth of flooding, because large numbers of houses in Chattanooga are on ground so low that second floors would be flooded and many one-story and a few two-story frame houses would be floated off their foundations, with almost complete loss of value. It was assumed that, on the average, 12 feet of water over the ground surface would enter the second story, that water deeper than 16 feet would float one-story frame houses, and that a depth of 23 feet or more would float two-story frame houses. Table 65 (table 6 on page 27 of original publication) shows the unit damage per room which applies to each class and kind of house for the depths of flooding mentioned above. Since the total number of rooms was estimated, whether one- or two-story houses, the per room damage was decreased approximately one-half for two-story houses flooded less than 12 feet. Building damage covered cost of reconstruction for depths above 16 feet for one-story frame houses and for two-story frame houses when flooded more than 23 feet.

Commercial damage—It was assumed that business establishments flooded would suffer practically a complete loss of stock and fixtures. This was the recent experience in Paducah and Louisville. While part of the loss in those cities was due to looting, such a loss appears unavoidable. Even where articles for sale received relatively small physical damage, their commercial value usually was negligible. The maximum flood stage in Chattanooga would be reached much sooner after the river passed above a flood stage than on the Ohio River at Louisville or Paducah, and less time would be available for salvage operations. Many of the Chattanooga stores have no direct access to an upper story; a number of good stores are in one-story buildings; and basements contain large stocks of goods and some salesrooms.

The amounts of estimated damage to the principal businesses usually varied between the following limits:

Auto sales	$ 2,000-$ 5,000
Auto repair	500- 2,000
Gasoline filling stations	300- 1,000
Barber shops	200- 1,000
Clothing stores	10,000- 50,000
Drug stores	10,000- 40,000
Furniture stores	10,000- 20,000
Grocery stores	1,500- 5,000
Hardware stores	5,000- 12,000
Restaurants	500- 4,000
Shoe stores	5,000- 40,000
Department stores, assuming ground floor area to include basement salesrooms (per square foot of floor space)	5- 12

A large number of small stores were appraised lower than the above limits.

The appraisal of damage to each business establishment was made by an appraiser with more than 20 years' experience in real-estate operation in Chattanooga in commercial and industrial property, who had personal knowledge of all parts of the commercial district and most of the individual business establishments.

Industrial damage—Each industry was given separate consideration. The floor space and floor elevations were obtained in most instances. The cooperation of the owner or his representative was sought in each industry. It was found that the damage to heavy industries like foundries and boiler shops was the lowest in proportion to the total value, while it was the highest in textile mills. Textile engineers in charge of full-fashioned hosiery mills appraised the flood damage to knitting machines costing about $10,000 each to be at least 50 percent. One establishment contained 44 such machines. The purchase or repair and installation of new textile machinery would require a cessation of operations of from 1 to 4 months.

Indirect damage—The indirect damage was appraised and computed for loss of industrial wages, loss of commercial and clerical wages, loss of industrial output (after deducting wages, material, fuel, and purchased energy), loss of profit on retail sales, loss of receipts by transportation companies and public utilities, and expenditures by relief agencies for those made homeless. This appraisal is as follows:

Industrial labor lost:		
18,000 at rate of $800 per year for average of 1 month	$1,200,000	
Additional 3,000 for 2 weeks	100,000	
		$1,300,000
Trade and clerical labor lost:		
12,000 at rate of $12.50 per week for 4 weeks	600,000	
Additional 6,000 at rate of $12.50 per week for 2 weeks	150,000	
		750,000
Loss of industrial output: Value of products less value of material, fuel, purchased energy, and labor (as shown in U. S. Industrial Census of Chattanooga—average of 1929 and 1935) for period of 1 month		1,750,000
Loss of retail sales profit: Gross sales shown in U. S. Census as $32,000,000 per year. Assuming net profit of 15 percent, or $4,800,000 per year, the loss for 1 month would be		400,000
Public Services and Utilities:		
City water works:		
Loss of business to 12,000 homes at $1	12,000	
Loss of business to industry	10,000	
		22,000
Power and light:		
Loss of sales to industry	200,000	
Loss of sales to homes	10,000	
Loss of bus receipts	20,000	
		230,000
Nashville, Chattanooga & St. Louis Ry: Loss of Revenue		30,000
Southern Ry.: Loss of revenue		60,000
Gas company:		
2,000 houses at $2.50	5,000	
Industrial customers	10,000	
		15,000
Telephone company: 1,000 houses for 1 month at $3		3,000
Relief agencies:		
Extra cost above ordinary living expenses for 30 days	1,500,000	
From homes that are destroyed for an additional 30 days	450,000	
		1,950,000
Total of all indirect losses		$6,510,000

TABLE 65 *(Table 6 in original publication).—Estimated flood damage per room, residential property.*

Depth of flooding, feet	Damage	Class I				Class II				Class III			
		Frame		Brick		Frame		Brick		Frame		Brick	
		1-story	2-story	1-story	2-story	1-story	2-story	1-story	2-story	1-story	2-story	1-story	2-story
1-12	Contents	$300	$150	$300	$150	$180	$ 75	$180	$ 75	$ 50	$30	$ 50	$30
	Building	100	50	100	50	90	45	90	45	50	30	50	30
	Total	400	200	400	200	270	120	270	120	100	60	100	60
12-16	Contents	300	300	300	300	180	180	180	180	50	50	50	50
	Building	100	100	100	100	90	90	90	90	50	50	50	50
	Total	400	400	400	400	270	270	270	270	100	100	100	100
16-23	Contents	300	300	300	300	180	180	180	180	50	50	50	50
	Building	800	100	140	100	600	90	120	90	300	50	60	50
	Total	1,100	400	440	400	780	270	300	270	350	100	110	100
23	Contents	300	300	300	300	180	180	180	180	50	50	50	50
	Building	800	800	150	120	600	600	120	100	300	300	70	60
	Total	$1,100	$1,100	$450	$420	$780	$780	$300	$280	$350	$350	$120	$110

APPLICATION OF APPRAISAL TO PAST FLOODS

By use of the information secured for each residential block, each commercial block, and each industry, the damage was appraised for stages 5 and 10 feet less than the 1867 stage. The indirect damage for a flood 5 feet less than in 1867 was assumed to be 60 percent of the damage at the 1867 stage; for a flood 10 feet less it was assumed to be 25 percent of the 1867 flood-stage figures. Damage begins at about 25 feet below the 1867 stage.

Table 66 (table 7 in original publication) shows the appraised flood damage for the 1867 flood stage and for stages 5 and 10 feet lower.

A flood equal to that of 1867 would flood 12,631 houses containing 52,800 rooms. Some houses are situated on land 25 feet below flood level. About 3,500 one-story frame houses would be flooded to a depth of 16 feet or more, resulting in almost complete destruction. The entire commercial district of the city would be flooded from 6 to 12 feet above the street levels. The flooded area would include 2,133 commercial establishments. Nearly all of the industrial establishments would be flooded, about

TABLE 66 *(Table 7 in original publication).—Appraised Chattanooga flood damages.*

Kind of property damaged	Flood stage		
	58 feet (1867)	53 feet	48 feet
Residential	$11,970,000	$6,958,000	$4,008,000
Commercial	8,160,000	3,467,000	1,017,000
Industrial	11,016,000	7,287,000	2,504,000
Direct damage	—	—	—
Subtotal	31,146,000	17,712,000	7,529,000
Indirect loss	6,510,000	3,900,000	1,500,000
Total	37,656,000	21,612,000	9,029,000

TABLE 67 *(Table 8 in original publication).—Estimated flood damages to Chattanooga as developed in 1938 caused by repetition of past floods.*

Year	Gage height, feet	Flood damage
1932	37.6	$1,000,000
1892	38.0	1,100,000
1880	38.4	1,200,000
1879	38.1	1,100,000
1902	38.0	1,100,000
1897	38.2	1,100,000
1883	38.5	1,200,000
1899	38.6	1,200,000
1926	38.4	1,200,000
1929	38.7	1,300,000
1891	38.9	1,400,000
1899	40.2	1,900,000
1882	40.4	2,000,000
1896	40.5	2,100,000
1902	40.8	2,300,000
1918	42.7	3,500,000
1890	42.6	3,500,000
1884	42.9	3,600,000
1920	43.6	4,100,000
1917	47.7	8,700,000
1886	52.2	19,000,000
1875	53.8	24,000,000
1867	57.9	37,600,000
	Total	$125,200,000

Length of period, 72 years.
Average damage per year, $1,739,000.

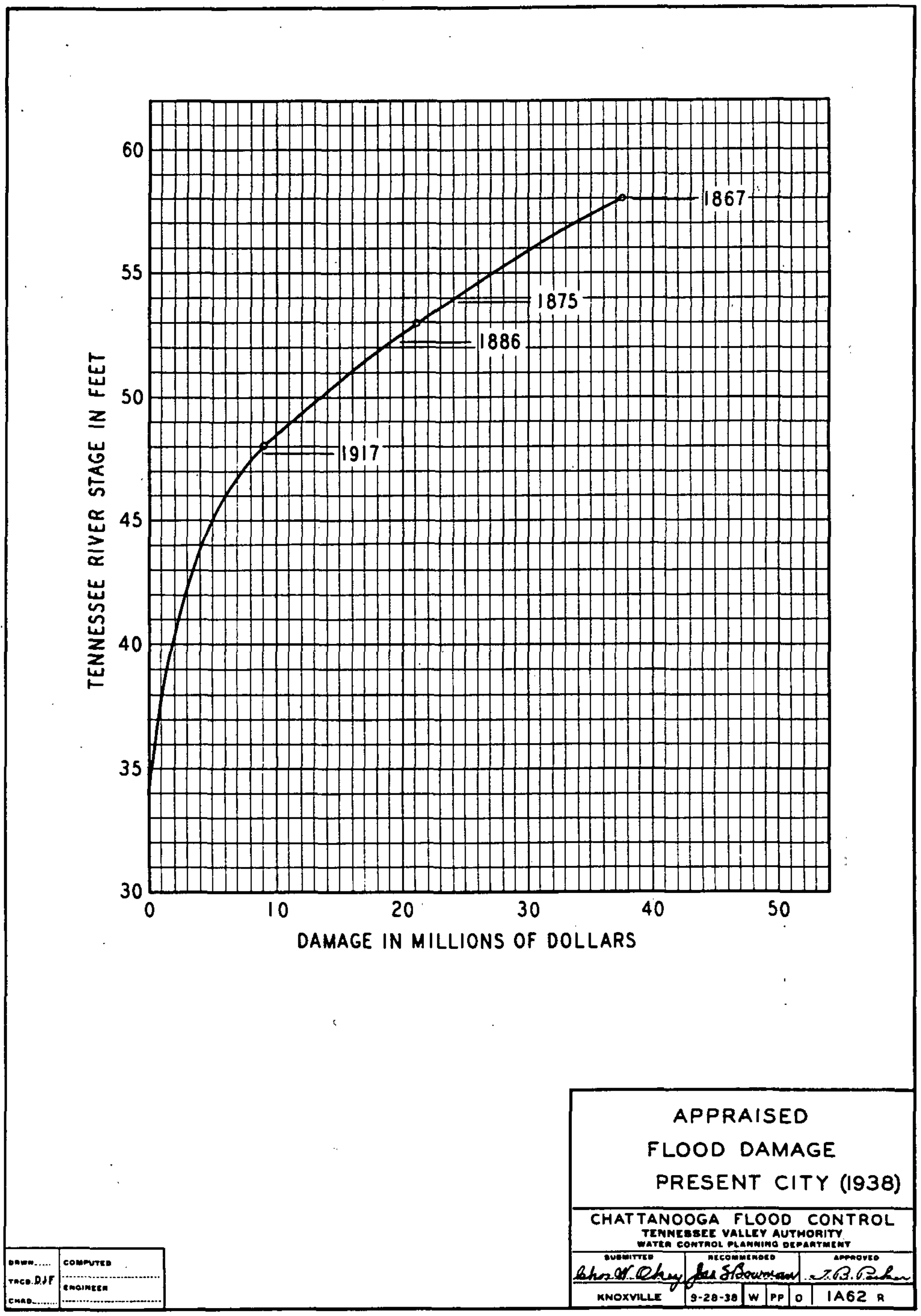


FIGURE 151 *(Plate 5 in original publication).—Appraised flood damage—city of Chattanooga 1938.*

half of them 12 feet or more above the ground floor. Many of them would be flooded to a depth of 20 feet.

The results of the appraisal at the three flood elevations are plotted in figure 151 (plate 5 in original publication). A smooth curve starting at river gage 34 feet with zero flood damage has been drawn through these four points. River stage records are continuous at Chattanooga from 1875. There may have been floods of moderate size between the maximum flood of 1867 and 1875. Flood stages from 1875 to 1938, large enough to cause damage, together with the corresponding damage to the present city determined from the curve on figure 151 (plate 5 in original publication) are shown in table 67 (table 8 in original publication).

Any smaller floods which may have occurred between 1867 and 1875, if included, would increase the totals slightly.

If only the floods of the 20-year period from 1867 to 1886 were considered, the total damage from them would be $89,700,000, an average of $4,985,000 per year. The results from so short a period should not be given much weight in considering the economics of flood-control expenditures, but they do indicate what has happened in one generation of Chattanooga history. A repetition of the 1867-86 floods in a similar length of time would seriously and permanently cripple the entire city unless protection is provided.

DAMAGES FROM "DESIGN" FLOOD

A flood 10 feet higher than that of 1867 would flood an additional 5,000 houses. A study of the information obtained for the individual blocks in the residence areas shows that 10,500 houses would be destroyed. Such a flood stage would reach the second floor of all the buildings in the commercial district and would flood all the industrial plants. The business and industrial life of the city would be interrupted for a much longer period, and the rehousing of at least 50,000 people whose homes were destroyed would require several months. For such a flood, the damage would be as follows:

Residential	$28,000,000
Industrial	18,000,000
Commercial	10,000,000
Indirect	14,000,000
Total	$70,000,000

INTANGIBLE FLOOD DAMAGE

The appraisal heretofore described considers only direct physical damage and indirect but measurable losses of wealth which the flood would cause. There are additional losses which always accompany the severe flooding of large centers of population which are difficult to measure but are nonetheless of great economic and social importance. These losses are usually termed "intangible," because they cannot be easily or accurately appraised or measured. The intangible damage may be as important to the future of a city as the physical damage.

The intangible damage of a flood of maximum proportions in Chattanooga would be unusually important. More than one-third of the population would be made homeless within a day or two after the lowest homes were flooded. The flood would occur probably in the period from January to March, before the cold weather was over. Exposure would be suffered by a large proportion of the homeless, with resulting sickness. Food stores would be destroyed and the supply of drinking water shut off. Since railway communication would be severed well beyond the city limits, the transportation of refugees and supplies would be confined to two highways, with most of the burden falling on the one to the south. The resulting congestion would result in increased hardships to the homeless. A large additional population in the flood-isolated homes would suffer from loss of light, power, heat, drinking water, and food. Even the parts of the city not isolated by the flood water would suffer almost as much hardship.

About 15,000 people would be driven from houses that were destroyed, presenting a rehousing problem of major proportions that could hardly be taken care of soon enough to prevent a large loss in population to the city. The reconstruction of these homes in the deeply flooded district would be inadvisable and extremely difficult to finance. Building sites above possible overflow, close enough to the industries to be suitable for workers' residences, are very limited in extent. Practically all the retail and wholesale merchants would lose their entire stock, and large numbers would face bankruptcy, since few retail merchants are free of loans secured by inventory.

The industrial development within the flooded area would be arrested. Good sites for industry outside the flood area are scarce or remote from the present city. There would be a tendency for the existing industries susceptible to great damage by floods to seek locations away from Chattanooga. The last flood which reached the business district of the city was in 1886, beyond the memory of nearly all the population and before the building of most of the present city. The moderate flood of 1917 affected only a part of the industrial district and practically none of the commercial area, but 10 years later the flood menace caused the city to lose to a neighboring city free from floods an industry planning to expend $10,000,000 for the building of a plant.

Many of these ill effects suffered from a great flood would be permanent, certainly as far as the present inhabitants are concerned. The loss would have a more permanent effect than one as great from fire; first, because there would be no insurance protection, and, second, because of the certainty of recurrence within a period of unpredictable length, possibly as short as 2 or 3 years.

The intangible injury to Chattanooga and the certainty of the recurrence of floods would place the city at a permanent disadvantage in competition with neighboring cities free from floods. Certain communities in the Mississippi Valley have, because of a flood menace, experienced a decline in industrial production and population. There appears no good reason why Chattanooga would not suffer a similar experience when floods as large or larger than those of the past recur.

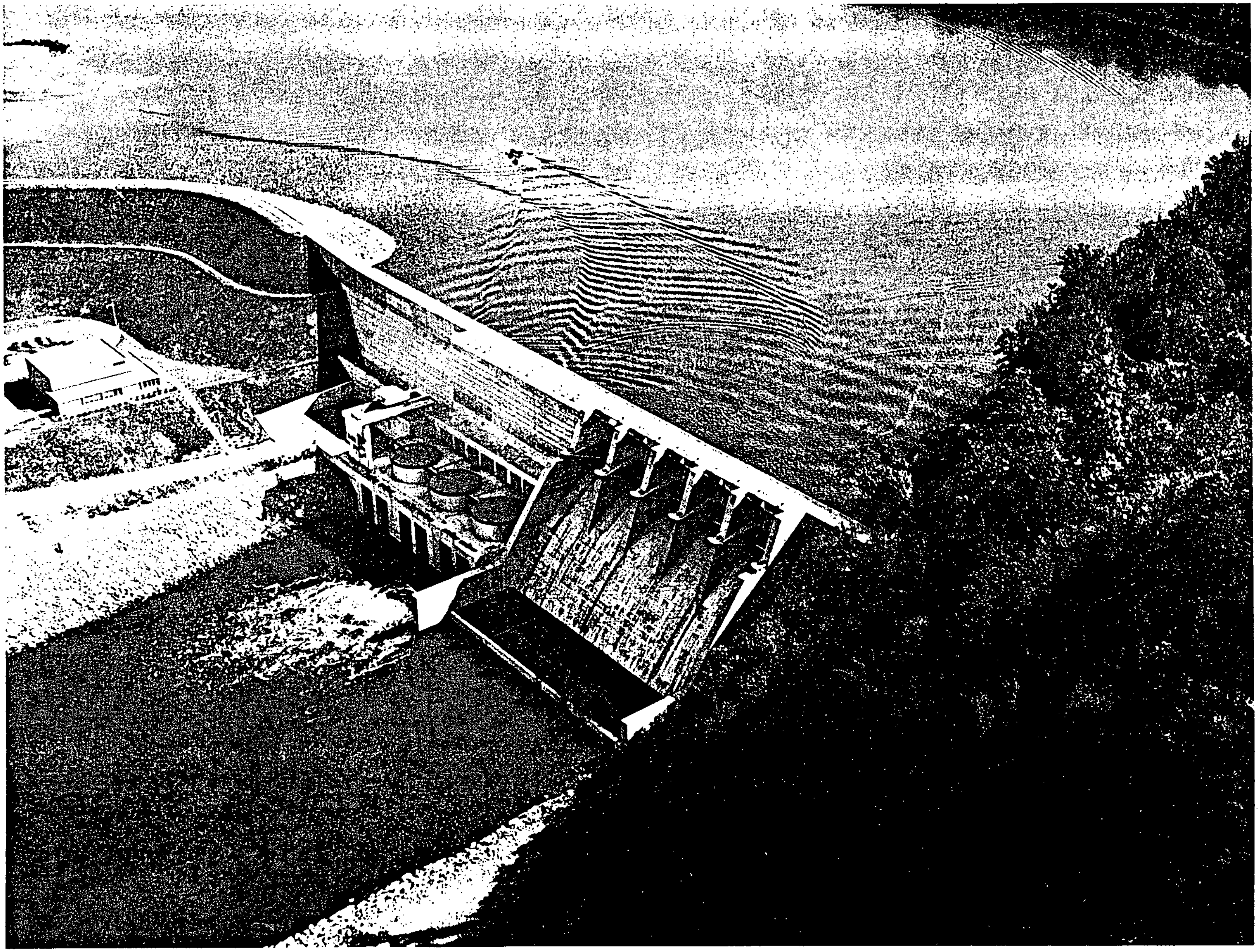


FIGURE 152.—*Boone Dam—TVA's latest (to 1961) multiple-purpose project to be placed in operation.*

APPENDIX D

REPORT ON ALLOCATION OF COSTS

AS OF JUNE 30, 1953,

PURSUANT TO SECTION 14 OF THE TENNESSEE VALLEY AUTHORITY ACT,

AND NOTES ON THAT ALLOCATION

This appendix includes TVA's most recent allocation report which was prepared—as of June 30, 1953—after completion of Boone, the latest (to 1961) multiple-purpose project to be placed in operation. This report, submitted to the President by the Board of Directors December 15, 1953, was approved by him January 21, 1955. Following the report itself are the notes on the 1953 allocation.

The report and notes are printed in their entirety although some small portion of their contents also appears in chapter 14.

Following is the letter submitting the report to the President:

Knoxville, Tenn.,
December 15, 1953.

The President,
The White House,
Washington, D. C.

My Dear Mr. President: As provided by Section 14 of the Tennessee Valley Authority Act, the Board of Directors of the Tennessee Valley Authority submits herewith its report on the allocation of the system investment as of June 30, 1953. A certified copy of a resolution of the Board adopting this report is attached hereto.[1]

Gordon R. Clapp
Chairman of the Board

Enclosures.

REPORT ON ALLOCATION OF COSTS AS OF JUNE 30, 1953, PURSUANT TO SECTION 14 OF THE TENNESSEE VALLEY AUTHORITY ACT

Section 14 of the Tennessee Valley Authority Act directs the TVA to allocate the investment in the dams, steam plants, or other similar improvements among (1) flood control, (2) navigation, (3) fertilizer, (4) national defense, and (5) the development of power. As new multiple-purpose projects have been added to the system, TVA has reported its allocations of investment to the President and to the Congress. The last report, dated November 19, 1951, presented the allocation of the system investment as of June 30, 1951.

In the fiscal year ended June 30, 1953, the TVA practically completed and placed in operation the Boone project on the South Fork Holston River in northeastern Tennessee. This addition brings the total number of multiple-purpose water control projects to nineteen, seventeen of which at present include generating units. In addition to the multiple-purpose hydro projects, the system includes nine single-purpose hydro plants; four large and modern steam-electric plants constructed by TVA; eight older and smaller steam-electric plants acquired by TVA; and other electric plant including transmission and switching facilities.

The total investment in this system as of June 30, 1953, amounting to $1,263,498,947, has been reviewed to determine the amounts to be allocated to the various purposes served. The allocation reported herein is based upon the principles outlined in the report published as House Document 709, 75th Congress, 3d session, entitled "Investment of the Tennessee Valley Authority in Wilson, Norris and Wheeler Projects."

The investment in multiple-purpose projects may be divided into two classes for purposes of allocation: (1) The investment in facilities used for a single purpose, such as powerhouses and generators provided for the production of power, sluiceways and portions of reservoirs to maintain flood control storage, or locks and river channel improvements provided for navigation. These costs are charged in their entirety to the purpose served. (2) Investment in facilities which serve more than one purpose, such as dams and the major portions of reservoirs. These costs are common to several purposes and so are divided among the purposes served.

The various projects in the system are so interrelated and interdependent in their operation that any practical assignment of costs of common facilities to their several uses can be made only by considering these projects as a system.

1. Board resolution is not printed in this volume (refers to report as submitted to the President).

Allocation of the common investment to navigation, flood control, and power is made in accordance with the percentages shown in the tabulation below, "Allocation of investment in multiple-purpose projects" (first table appearing on page 5 in report as submitted to the President). No allocation is made to the fertilizer program because it is charged for the power provided by the TVA system for this program. Similarly, no allocation is made to national defense although large amounts of power are being supplied to defense establishments and extra costs have been incurred due to accelerated construction schedules during periods of national emergency.

The following tabulation shows the distribution of the investment as of June 30, 1953. Properties constructed or developed by TVA are stated at cost to TVA. The amount shown for the Wilson project is the estimated reconstruction-cost-new of the original project at the date of its transfer to TVA (June 16, 1933), plus the cost of additions less retirements made by TVA to June 30, 1953. Properties purchased from utility companies are stated on the basis of their original cost when first devoted to public service, plus cost of additions less retirements.

Project	Date in service for TVA system use	Investment as of June 30, 1953
Multiple-purpose projects (costs subject to allocation)		
Wilson (acquired)	June 16, 1933	$ 47,371,548
Norris	July 28, 1936	30,063,896
Wheeler	November 9, 1936	45,908,380
Pickwick Landing	June 29, 1938	42,022,799
Guntersville	August 1, 1939	34,292,976
Hales Bar (acquired)	August 16, 1939	29,604,185
Chickamauga	March 4, 1940	36,917,712
Hiwassee	May 21, 1940	16,609,768
Nottely	February 1, 1942	5,377,814
Watts Bar	February 11, 1942	32,583,837
Chatuge	March 1, 1942	7,037,848
Multiple-purpose projects (costs subject to allocation)—Continued		
Cherokee	April 17, 1942	32,428,735
Douglas	March 21, 1943	41,467,960
Fort Loudoun	November 9, 1943	39,034,076
Kentucky	September 14, 1944	114,382,302
Fontana	January 20, 1945	70,134,425
Watauga	August 30, 1949	31,615,729
South Holston	February 13, 1951	30,555,211
Boone	March 16, 1953	21,087,987
Navigation Channel Improvements		10,254,270
Subtotal		$ 718,751,458
Single-purpose projects (cost charged to power)		
Hydroelectric Projects, Constructed by TVA		
Ocoee No. 3	April 30, 1943	$ 7,984,111
Apalachia	September 22, 1943	22,269,085
Subtotal		$ 30,253,196
Hydroelectric Projects, Acquired by TVA		
7 Plants		$ 18,505,252
Steam-Electric Plants, Constructed by TVA		
Watts Bar	February 15, 1941	$ 18,975,824
Johnsonville	October 27, 1951	84,775,340
Widows Creek	July 1, 1952	57,433,542
Shawnee	April 9, 1953	51,775,385
Subtotal		$ 212,960,091
Steam-Electric Plants, Acquired by TVA		
8 Plants		$ 14,166,571
Other electric plant (costs charged to power)		
Transmission lines, substations, general plant, and land		$ 268,862,379
Total Investment		$1,263,498,947

The allocation of the investment as of June 30, 1953, in multiple-purpose projects and the allocation of the total system investment including single-purpose projects and other electric plant is shown in the following tables (tabulations below).

Allocation of investment in multiple-purpose projects

Purpose	Direct costs	Allocation of common costs: Percent	Allocation of common costs: Amount	Total: Amount	Total: Percent
Navigation	$ 44,938,051	27.0	$110,646,189	$156,580,240	21.8
Flood control	55,367,000	31.0	127,038,217	182,405,217	25.4
Power	207,649,707	42.0	172,116,294	379,766,001	52.8
Total	$308,950,758	100.0	$409,800,700	$718,751,458	100.0

Allocation of total system investment

Purpose	Allocated investment in multiple-purpose hydro projects	Single-purpose hydro, steam-electric, and other electric plant	Total: Amount	Total: Percent
Navigation	$156,580,240	—	$ 156,580,240	12.4
Flood control	182,405,217	—	182,405,217	14.4
Power	379,766,001	$544,747,489	924,513,490	73.2
Total	$718,751,458	$544,747,489	$1,263,498,947	100.0

The above percentages for the allocation of common costs are applied only to investment in the system of multiple-purpose projects as of June 30, 1953. New allocation reports will be submitted whenever the system characteristics are changed by the addition of multiple-purpose projects.

NOTES ON ALLOCATION OF THE INVESTMENT OF THE TENNESSEE VALLEY AUTHORITY AS OF JUNE 30, 1953—19-PLANT MULTIPLE-PURPOSE SYSTEM

Introduction

The Congressional Act creating the Tennessee Valley Authority directs the TVA to allocate the investment in the various projects of the system to the purposes served. It specifically lists (1) flood control, (2) navigation, (3) fertilizer, (4) national defense, and (5) production of power as purposes to which allocations of cost should be made.

In 1938 the TVA Committee on Financial Policy, acting with the advice of qualified consultants, made an extensive review and exploration of various allocation methods and theories. These are discussed in detail in the report published as House Document No. 709, 75th Congress, 3d Session, entitled "Investment of the Tennessee Valley Authority in Wilson, Norris and Wheeler Projects." The Committee concluded that no one method furnished a completely satisfactory basis for cost allocation. Among those considered, however, the "alternative-justifiable-expenditure" method was believed to be most workable and to furnish a fair guide for determining the relative shares of the various functions in the joint costs. The allocations made at that time and in subsequent years have, in general, followed this method.

The TVA system of multiple-purpose water control projects has been designed to provide for navigation, flood control, and power generation. Many years of operating experience has demonstrated the effectiveness of the system in fulfilling these objectives. As new multiple-purpose projects have been added to the system, TVA has reported its allocation of investment to the President and to the Congress. In these allocation reports the investment has been allocated to navigation, flood control, and power. No allocation has been made to the fertilizer program because it is charged for the power it obtains from the multiple-purpose system. Nor is any portion of the investment allocated to national defense, although the system is a great national defense asset, and although the extra cost of building several war emergency projects on accelerated schedules and the low sale price of power furnished by the system in large amounts to Federal defense agencies of uncertain duration might logically have been considered to provide a basis for a charge against national defense.

Congressional committees on two occasions have reviewed TVA allocation procedures. In 1938, the Congress established a "Joint Committee on the Investigation of the Tennessee Valley Authority." This committee and its staff devoted a great amount of time and effort to the problem. The allocation was approved by the committee after a lengthy engineering analysis and a thorough investigation. The report of this committee, published as Senate Document No. 56, 76th Congress, 1st Session, states:

> The committee's engineers comment at length on the problems of allocating the common investment in the Authority's program. They report that the Authority made an extensive study taking into consideration all of the factors involved; and in their opinion, while certain minor considerations could have been taken into account, they adopted the Authority's figures as a basis for the study of its wholesale rates, "as being within reasonable limits of what a group of impartial engineers and economists would determine."

Ten years later, in 1948, a Subcommittee of the Senate Committee on Public Works requested a review of the Authority's allocations by the Federal Power Commission. The commission was assisted in this review by the Corps of Engineers and the General Accounting Office. The principal points considered in this review were the principles and methods used in the TVA allocations—the correctness of the application of such principles and methods, and the economic justification as related to the amounts allocated to specific purposes. The conclusions reached by the Federal Power Commission and reported to the Public Works Subcommittee are as follows:

A. The alternative single-purpose projects are justifiable on a cost-benefit ratio basis, and the use of the estimated costs of such projects in arriving at the percentages for allocating joint costs is reasonable;
B. The principles and methods employed by TVA in allocating the joint costs of its multiple-purpose projects are reasonably adapted for the purpose;
C. The allocation of the actual joint costs of the multiple-purpose projects to navigation, flood control, and power made by TVA in its report dated November 13, 1945, is reasonable and should be accepted for the purposes of the Tennessee Valley Authority Act.

The principles and methods used in determining the allocation of the common costs for the fiscal year ending on June 30, 1953, are consistent with those previously used.

A list of official TVA reports and additional references to publications concerning TVA allocations is attached at the conclusion of these notes.

Investment as of June 30, 1953

In the fiscal year ending June 30, 1953, the Tennessee Valley Authority put into operation the multiple-purpose Boone hydro project located on the South Fork Holston River in northeastern Tennessee. This project represents the only major addition to the system investment in multiple-purpose facilities since the last allocation report of 1951. Single-purpose additions to the water control system since 1951 include the extension of the Hales Bar powerhouse with the installation of units 15 and 16, and the installation of generating units in vacant stalls at several existing hydro plants. The Johnsonville, Widows Creek, and Shawnee steam-electric plants were also put into operation during this interval.

The distribution of the investment as of June 30, 1953, in multiple-purpose projects is shown in table 68 (table I in report as submitted to the President). Properties constructed or developed by TVA are stated at cost to TVA. The amount shown for the Wilson project is the estimated reconstruction-cost-new of the original project at June 16, 1933, plus the cost of additions less retirements made by TVA to June 30, 1953. Properties purchased from utility companies are stated at their original cost when first devoted to public service, plus the cost of additions less retirements.

The investment in single-purpose hydro and steam power plants and other electric plant, including transmission lines and switching facilities, is shown in table 69 (table II in report as submitted to the President).

The total investment in plant in service as of June 30, 1953, amounted to $1,263,498,947.

The alternative-justifiable-expediture method

The application of the alternative-justifiable-expenditure method as a guide to the allocation of the TVA system investment involves the following steps:

1. Identify the costs subject to allocation. These include the entire investment in multiple-purpose projects on June 30, 1953, and also the estimated expenditures necessary to complete the work in progress at such projects. The investment in single-purpose hydro projects, steam-electric plants, transmission facilities, and substations is not involved in this application of the alternative-justifiable-expenditure method because it is allocated entirely to power.
2. Determine the direct costs for each function.

TABLE 68 *(Table I in report as submitted to the President).—Investment in multiple-purpose projects.*

Project	Date in service for TVA system use	System as of June 30, 1953: No. of generating units[1]	System as of June 30, 1953: Investment[2] subject to allocation	Estimated total cost of work in progress	System upon completion of work in progress: No. of generating units[1]	System upon completion of work in progress: Investment
Wilson (acquired)	June 16, 1933	18	$ 47,371,548	$ 190,036	18	$ 47,561,584
Norris	July 28, 1936	2	30,063,896	40,767	2	30,104,663
Wheeler	November 9, 1936	8	45,908,380	239,510	8	46,147,890
Pickwick Landing	June 29, 1938	6	42,022,799	0	6	42,022,799
Guntersville	August 1, 1939	4	34,292,976	867,760	4	35,160,736
Hales Bar (acquired)	August 16, 1939	16	29,604,185	1,182,551	16	30,786,736
Chickamauga	March 4, 1940	4	36,917,712	138,868	4	37,056,580
Hiwassee	May 21, 1940	1	16,609,768	5,381,418	2	21,991,186
Nottely	February 1, 1942	0	5,377,814	2,538,413	1	7,916,227
Watts Bar	February 11,1942	5	32,583,837	98,422	5	32,682,259
Chatuge	March 1, 1942	0	7,037,848	2,307,717	1	9,345,565
Cherokee	April 17, 1942	3	32,428,735	2,711,814	4	35,140,549
Douglas	March 21, 1943	3	41,467,960	3,237,992	4	44,705,952
Fort Loudoun	November 9, 1943	4	39,034,076	22,735	4	39,056,811
Kentucky	September 14, 1944	5	114,382,302	94,817	5	114,477,119
Fontana	January 20, 1945	2	70,134,425	3,389,354	3	73,523,779
Watauga	August 30, 1949	2	31,615,729	0	2	31,615,729
South Holston	February 13, 1951	1	30,555,211	41,730	1	30,596,941
Boone	March 16, 1953	2	21,087,987	3,123,498	3	24,211,485
Navigation Channel Improvements		—	10,254,270	202,982	—	10,457,252
Total multiple-purpose projects			$718,751,458	$25,810,384		$744,561,842

1. Excludes station service units.
2. Financial Statements for June 30, 1953, Schedule A.

TABLE 69 *(Table II in report as submitted to the President).* *—Investment in single-purpose plant.*

Costs charged to power			
Project	Date in service for TVA system use	No. of generating units[1]	Investment as of June 30, 1953[2]
Hydroelectric projects, constructed by TVA			
Ocoee No. 3	April 30, 1943	1	$ 7,984,111
Apalachia	September 22, 1943	2	22,269,085
Subtotal			$ 30,253,196
Hydroelectric projects, acquired by TVA			
Blue Ridge	August 16, 1939	1	$ 4,909,113
Ocoee No. 1	August 16, 1939	5	2,369,408
Ocoee No. 2	August 16, 1939	2	2,611,878
Great Falls	August 16, 1939	2	4,655,262
Columbia	August 16, 1939	2	207,918
Nolichucky	June 30, 1945	4	1,471,351
Wilbur	June 30, 1945	4	2,280,322
Subtotal			$ 18,505,252
Steam-electric plants, constructed by TVA			
Watts Bar	February 15, 1942	4	$ 18,975,824
Johnsonville	October 27, 1951	6	84,775,340
Widows Creek	July 1, 1952	4	57,433,542
Shawnee	April 9, 1953	2	51,775,385
Subtotal			$212,960,091
Steam-electric plants, acquired by TVA			
Hales Bar	August 16, 1939	2	$ 3,286,928
Nashville	August 16, 1939	6	$ 3,062,707
Wilson	October 10,1939	3	561,038
Memphis	May 31,1950		4,241,149
4 Small Plants			3,014,749
Subtotal			$ 14,166,571
Other electric plant			
Transmission lines, substations, and general plant and land			$268,862,379
Total single-purpose plant			$544,747,489

1. Excludes station service units.
2. *Financial Statements for June 30, 1953,* Schedules B and C.

3. Determine common costs by deducting the total of the direct costs from the total investment subject to allocation.
4. Estimate the alternative justifiable expenditure for three separate, single-use systems designed for navigation, flood control, and power that would provide benefits equivalent to those provided by the multiple-purpose system of projects. The alternative expenditure must not exceed the value of the benefits.
5. Deduct the direct costs from the estimated alternative justifiable expenditures to establish the remaining alternative justifiable expenditure for each function. These remaining costs represent the maximum expenditure that could be economically justified as common costs for each function in the multiple-purpose system.
6. Find the percentages that the remaining alternative justifiable expenditure for each function bears to the total of such expenditures.
7. The percentages thus obtained are then used as a basis for judgment in determining the final percentage of common investment to be allocated to each purpose.

The allocation of the system investment as of June 30 is obtained by applying the adopted percentages to the common investment and adding to these allocated amounts the direct cost for each purpose.

The following notes outline each step in some detail and furnish the supporting data for the calculations in table 77 (table X in report as submitted to the President).

Investment subject to allocation

The total system cost, used in the determination of the percentages for allocation of the common costs, has been obtained by adding the estimated total cost of completing the work progress on June 30, 1953, to the investment as of that date. The distribution of this cost to the multiple-purpose projects is shown in table 68 (table I in report as submitted to the President).

For allocation purposes, the investment in TVA multiple-purpose projects may be divided into two general classes, described in more detail in the following paragraphs: (1) The investment in facilities used for only one purpose, such as powerhouses and generating equipment provided for the production of power, sluiceways and the portions of reservoirs which provide space for flood control purposes, or locks and river channel improvements used for navigation. These costs are charged in their entirety to the purpose served. (2) Investment in facilities which serve more than one purpose, such as dams and the portions of reservoirs which provide space used for more than a single purpose. These costs are divided among the purposes served.

Direct investment

The direct investment for any one purpose in a multiple-purpose project is considered to be the investment in facilities used for only that purpose and which could have been eliminated had that purpose been excluded, at the same time leaving a complete structure to provide for the proper functioning of the remaining purposes.

Navigation—The direct investment chargeable to navigation at any project is the cost of the facilities useful only for navigation, such as the lock and lock machinery, less the estimated cost of a section of dam necessary to replace the lock and to leave a complete structure. The cost of channel improvements is also considered as a direct investment for navigation.

Flood control—The direct flood control investment at any project is the cost of facilities specifically provided for that purpose, such as sluiceways, and includes the cost of increased height of dam and reservoir facilities necessary to provide storage space in addition to that normally required for the other purposes. Such costs may be determined by deducting the estimated cost of a theoretical dual-purpose project, designed for navigation and power, from the cost of the multiple-purpose project as constructed.

At each main-river project, the height of a dual-purpose structure at that site is fixed by the normal maximum operating level for navigation and power under multiple-purpose operating schedules. At each storage project located on the tributaries, the height of a dual-purpose structure at that site is determined by the average of the maximum elevations to which the multiple-purpose reservoir fills annually for navigation and power purposes after observing rules which limit filling during the flood season. These levels are shown in table 70 (table III in report as submitted to the President). The elevations to which the tributary reservoirs will fill are influenced by the levels permitted during the flood season; therefore, elevations on two selected dates occurring during the flood season are also stated in table 70 (table III in report as submitted to the President).

Power—The direct investment chargeable to power at each of the main-river projects and the Boone project is the cost of the power facilities less the cost of a section of dam that would be needed to replace the integral powerhouse-intake structure if it had been omitted from the project. At the other projects where the powerhouse is not a part of the dam structure, the direct cost of power is the total cost of the power facilities.

Direct and common costs

Table 71 (table IV in report as submitted to the President) gives the direct and common costs for each multiple-purpose project for plant in service as of June 30, 1953. The corresponding costs based upon the investment after completion of the work in progress are shown by table 72 (table V in report as submitted to the President).

Alternative justifiable expenditures or alternative cost

Estimates of cost are made of the most economical system of single-purpose structures which would furnish substantially the same quantity and quality of service for the single purpose as that provided by the multiple-purpose system. As most of the alternative projects were assumed to be built at sites of actual multiple-purpose projects, the estimates

TABLE 70 *(Table III in report as submitted to the President).—Reservoir elevations.*

Project	Elevation top of gates	Flood season: Normal multiple-purpose operating levels		After the flood season: Normal maximum operating levels main-river dams	After the flood season: Average maximum elevation reached annually tributary dams
Main-river projects					
Kentucky	375	354		359	
Pickwick Landing	418	408		414	
Wilson	507.88	504.5		507.5	
Wheeler	556.3	550		556	
Guuntersville	595.44	593		595	
Hales Bar	635	632		634	
Chickamauga	685.44	675		682.5	
Watts Bar	745	735		741	
Fort Loudoun	815	807		813	
Tributary projects		On January 1	On March 15		
Norris	1034	978	990		1005
Cherokee	1075	1020	1042		1061
Douglas	1002	935	958		986
Fontana	1710	1615	1644		1682
Hiwassee	1526.5	1455	1472		1515
Chatuge	1928	1910	1916		1922
Nottely	1780	1743	1755		1770
Watauga	1975[1]	1934	1952		1950
South Holston	1742[1]	1702	1713		1720
Boone	1385	1358	1375		1385

1. Spillway crest.

generally reflect actual knowledge of construction conditions. Estimates for single-purpose projects were based on construction cost levels experienced at the time of construction of the corresponding multiple-purpose project.

Navigation—The navigable channel created by the multiple-purpose system provides a minimum depth of 11 feet and a minimum width of 300 feet from Knoxville to the Ohio River, 650 river miles with a sailing line distance of approximately 630 miles. It was determined that the cheapest and best alternative way to accomplish this would be by the construction of a system of ten high dams. These dams would be located at the sites occupied by the present structures except on the lower river below Pickwick Landing project. It was estimated that two dams would provide cheaper single-purpose navigation in this reach of the river than a single dam located at the present site of Kentucky Dam.

TABLE 71 *(Table IV in report as submitted to the President).—Direct and common costs—June 30, 1953, investment.*

	Navigation			Flood control	Power			Common		
Project	Navigation facilities	Deduct replacement section	Direct costs	Direct costs	Power facilities	Deduct replacement section	Direct costs	Total investment	Less direct costs	Common costs
Wilson	$ 2,710,760	$ 125,000	$ 2,585,760	0	$ 29,198,769	$ 3,900,000	$ 25,298,769	$ 47,371,548	$ 27,884,529	$ 19,487,019
Norris	0	200,000	0	$ 5,506,000	4,975,738	—	4,975,738	30,063,896	10,481,738	19,582,158
Wheeler	1,920,987[1]	—	1,720,987	0	22,012,316	830,000	21,182,316	45,908,380	22,903,303	23,005,077
Pickwick Landing	5,813,399	380,000	5,433,399	788,000	22,775,511	550,000	22,225,511	42,022,799	28,446,910	13,575,889
Guntersville	3,281,547	440,000	2,841,547	0	12,872,761	780,000	12,092,761	34,292,976	14,934,308	19,358,668
Hales Bar	2,670,677	140,000	2,530,677	0	15,443,461	900,000[2]	14,543,461	29,604,185	17,074,138	12,530,047
Chickamauga	4,656,116	790,000	3,866,116	1,107,000	14,046,271	1,470,000	12,576,271	36,917,712	17,549,387	19,368,325
Hiwassee	0	—	0	1,356,000	3,446,711	—	3,446,711	16,609,768	4,802,711	11,807,057
Nottely	0	—	0	587,000	—	—	—	5,377,814	587,000	4,790,814
Watts Bar	3,100,963	565,000	2,535,963	1,952,000	13,253,556	614,000	12,639,556	32,583,837	17,127,519	15,456,318
Chatuge	0	—	0	501,000	—	—	—	7,037,848	501,000	6,536,848
Cherokee	0	—	0	3,467,000	7,648,716	—	7,648,716	32,428,735	11,115,716	21,313,019
Douglas	0	—	0	7,057,000	8,567,416	—	8,567,416	41,467,960	15,624,416	25,843,544
Fort Loudoun	5,625,059	940,000	4,685,059	786,000	13,754,276	1,610,000	12,144,276	39,034,076	17,615,335	21,418,741
Kentucky	9,690,273	210,000	9,480,273	16,532,000	22,127,813	2,890,000	19,237,813	114,382,302	45,250,086	69,132,216
Fontana	0	—	0	7,623,000	9,630,816	—	9,630,816	70,134,425	17,253,816	52,880,609
Watauga	0	—	0	3,045,000	8,201,655	—	8,201,655	31,615,729	11,246,655	20,369,074
South Holston	0	—	0	4,950,000	5,066,149	—	5,066,149	30,555,211	10,016,149	20,539,062
Boone	0	—	0	110,000	9,331,772	1,160,000	8,171,772	21,087,987	8,281,772	12,806,215
Navigation Channel Improvements	10,254,270	—	10,254,270	—	—	—	—	10,254,270	10,254,270	0
Total	$49,724,051	$3,790,000	$45,934,051	$55,367,000	$222,353,707	$14,704,000	$207,649,707	$718,751,458	$308,950,758	$409,800,700

1. Includes difference in investment of $104,523 between a high- and low-beam bridge.
2. In use for accounting purposes after July 10, 1952.

TABLE 72 *(Table V in report as submitted to the President).—Direct and common costs—system upon completion of work in progress.*

	Navigation			Flood control	Power			Common		
Project	Navigation facilities	Deduct replacement section	Direct costs	Direct costs	Power facilities	Deduct replacement section	Direct costs	Total investment	Less direct costs	Common costs
Wilson	$ 2,737,760	$ 125,000	$ 2,612,760	0	$ 29,361,059	$ 3,900,000	$ 25,461,059	$ 47,561,584	$ 28,073,819	$ 19,487,765
Norris	0	—	0	$ 5,506,000	5,003,462	—	5,003,462	30,104,663	10,509,462	19,595,201
Wheeler	1,920,987[1]	200,000	1,720,987	0	22,009,180	830,000	21,179,180	46,147,890	22,900,167	23,247,723
Pickwick Landing	5,813,399	380,000	5,433,399	788,000	22,736,011	550,000	22,186,011	42,022,799	28,407,410	13,615,389
Guntersville	3,281,547	440,000	2,841,547	0	12,885,096	780,000	12,105,096	35,160,736	14,946,643	20,214,093
Hales Bar	2,670,677	140,000	2,530,677	0	16,548,102	900,000	15,648,102	30,786,736	18,178,779	12,607,957
Chickamauga	4,656,116	790,000	3,866,116	1,107,000	14,096,794	1,470,000	12,626,794	37,056,580	17,599,910	19,456,670
Hiwassee	0	—	0	1,356,000	8,576,558	—	8,576,558	21,991,186	9,932,558	12,058,628
Nottely	0	—	0	623,000	2,581,000	—	2,581,000	7,916,227	3,204,000	4,712,227
Watts Bar	3,102,962	565,000	2,537,962	1,952,000	13,281,432	614,000	12,667,432	32,682,259	17,157,394	15,524,865
Chatuge	0	—	0	537,000	2,330,000	—	2,330,000	9,345,565	2,867,000	6,478,565
Cherokee	0	—	0	3,467,000	10,296,382	—	10,296,382	35,140,549	13,763,382	21,377,167
Douglas	0	—	0	7,057,000	11,758,513	—	11,758,513	44,705,952	18,815,513	25,890,439
Fort Loudoun	5,627,059	940,000	4,687,059	786,000	13,760,451	1,610,000	12,150,451	39,056,811	17,623,510	21,433,301
Kentucky	9,690,273	210,000	9,480,273	16,532,000	22,129,337	2,890,000	19,239,337	114,477,119	45,251,610	69,225,509
Fontana	0	—	0	7,623,000	13,079,523	—	13,079,523	73,523,779	20,702,523	52,821,256
Watauga	0	—	0	3,045,000	8,187,377	—	8,187,377	31,615,729	11,232,377	20,383,352
South Holston	0	—	0	4,950,000	5,068,881	—	5,068,881	30,596,941	10,018,881	20,578,060
Boone	0	—	0	110,000	11,794,641	1,160,000	10,634,641	24,211,485	10,744,641	13,466,844
Navigation Channel Improvements	10,457,252	—	10,457,252	—	—	—	—	10,457,252	10,457,252	0
Total	$49,958,032	$3,790,000	$46,168,032	$55,439,000	$245,483,799	$14,704,000	$230,779,799	$744,561,842	$332,386,831	$412,175,011

1. Includes difference in investment of $104,523 between a high- and low-beam bridge.

TABLE 73 *(Table VI in report as submitted to the President).—Alternative justifiable expenditure for navigation.*

Location of alternative single-use structure	Elevation of top of spillway gates	Estimated cost of project
Alternate for Kentucky		
Dam at river mile 39.6	333	$ 32,000,000
Dam at river mile 114.0	360	28,000,000
Pickwick Landing	408	16,899,000
Wilson	507.88	22,113,000
Wheeler	550	20,278,000
Guntersville	593	20,978,000
Hales Bar	635	14,975,000
Chickamauga	675	19,747,000
Watts Bar	735	14,873,000
Fort Loudoun	807	23,838,000
Channel improvements		9,117,000
Downstream benefits[1]		9,000,000
Total		$231,818,000

1. This amount represents the capitalized annual saving in navigation costs on the Mississippi River resulting from low-water releases from tributary dams located at Fontana, Norris, Hiwassee, Chatuge, Nottely, Cherokee, Douglas, Watauga, South Holston, and Boone.

Low-water releases from the tributary reservoirs are of benefit to navigation on the Mississippi River. Since it is considered that the alternative cost of providing tributary storage capacity for this purpose would not be justified by the benefits, the alternative justifiable expenditure for storage was limited to the capitalized value of these benefits measured in terms of the annual saving in navigation costs.

The alternative justifiable expenditure for the single-purpose navigation system is shown in table 73 (table VI in report as submitted to the President).

Flood control—The alternative single-use system for flood control includes reservoirs on the main river and major tributaries to provide flood control storage equal in amount and effectiveness to that provided by the multiple-purpose system. Each of these single-use projects would have flood storage capacity equivalent to that contained in from one to three of the multiple-purpose projects as built. The location, amount of storage provided, and the estimated cost of the alternative projects are shown in table 74 (table VII in report as submitted to the President). The elevation of the top of spillway gates for each of these hypothetical structures is also shown. The average cost per acre-foot for flood control storage in the alternative single-use system is obtained and applied to the actual amount of storage capacity available in the multiple-purpose system, as shown in table 75 (table VIII in report as submitted to the President) to determine the alternative justifiable expenditure for flood control.

Power—The alternative cost of power is based on the unit cost of power determined from a system of single-use plants capable of producing substantially the same amounts of primary and high-grade secondary power as that available from the multiple-purpose hydro system. The system of alternative single-use projects shown in table 76 (table IX in report as submitted to the President) was used as the basis of this unit cost.

TABLE 74 *(Table VII in report as submitted to the President).—Estimated cost of storage in single-use flood control system.*

Location of alternative single-use structure	Flood control storage available, acre-feet	Elevation of top of spillway gates	Estimated cost of project
Tributary sites			
Norris	1,635,000	1007.5	$ 19,000,000
Hiwassee	398,000	1520	12,501,000
Fontana	771,000	1632	27,532,000
Nottely	110,000	1758.4	4,532,000
Cherokee	1,146,000	1060	21,302,000
Douglas	1,311,000	995	30,694,000
Watauga	260,000	1900[1]	15,044,000
South Holston	400,000	1691.5[1]	18,125,000
Main river sites			
Watts Bar	844,000	745	16,538,000
Wheeler	541,000	545	17,348,000
Kentucky	4,477,000	375	78,554,000
Total	11,893,000		$261,170,000

Average cost per acre-foot—$22.00

1. Spillway crest.

TABLE 75 *(Table VIII in report as submited to the President).—Alternative justifiable expenditure for flood control.*

Name of multiple-use project	Elevation of top of spillway gates	January 1 elevation, flood schedule	Flood control storage available, acre-feet
Kentucky	375	354	4,011,000
Pickwick Landing	418	408	418,000
Wilson	507.88	504.5	53,000
Wheeler	556.3	550	349,000
Guntersville	595.44	593	163,000
Hales Bar	635	—	0
Chickamauga	685.44	675	329,000
Watts Bar	745	735	378,000
Fort Loudoun	815	807	109,000
Norris	1034	978	1,635,000
Watauga	1975[1]	1934	260,000
South Holston	1742[1]	1702	300,000
Cherokee	1075	1020	1,146,000
Douglas	1002	935	1,311,000
Fontana	1710	1615	771,000
Hiwassee	1526.5	1455	291,000
Chatuge	1928	1910	105,000
Nottely	1780	1743	110,000
Boone	1385	1358	100,000
Total			11,839,000

Alternative justifiable expenditure for flood control
11,839,000 x $22.00 = $260,458,000

1. Spillway crest.

The unit cost of power thus obtained, which represents the unit cost of potentially available primary and high-grade secondary hydro power, is applied to the corresponding potential output of the multiple-purpose system. However, the installed capacity in the multiple-purpose system upon completion of the work now in progress will be in excess of that necessary to develop the potential primary and high-grade secondary hydro power available. This is recognized and compensated for by adding the cost of units installed or under construction on June 30, 1953, which are in addition to the basic capacity requirements of the potential hydro system and are required to meet actual load demands on the combined hydro and steam system. Table 76 (table IX in report as submitted to the President) summarizes these computations.

TABLE 76 *(Table IX in report as submitted to the President).—Alternative justifiable expenditure for power.*

Alternative single-use system

Project	Maximum reservoir elevation	No. of units	Installed capacity, kw	Estimated cost
Pickwick Landing	418	5	180,000	$ 30,548,000
Wilson	507.88	18	436,000	43,365,000
Wheeler	556.3	8	259,200	39,933,000
Guntersville	595.44	4	97,200	30,691,000
Hales Bar	631	14	51,100	13,699,000
Chickamauga	685.44	4	108,000	34,157,000
Watts Bar	745	5	150,000	30,376,000
Fort Loudoun	815	2	64,000	28,568,000
Norris	1034	3	120,000	31,600,000
Cherokee	1075	3	90,000	30,783,000
Douglas	1002	3	90,000	40,500,000
Fontana	1710	3	202,500	72,190,000
Hiwassee	1526.5	1	57,600	16,709,000
Chatuge	1928	0	0	7,197,000
Total			1,905,600	$450,316,000

Primary power	931,000 continuous kw	
Secondary power—222,000 kw Equivalent primary power (3/8 x 222,000)	83,000 continuous kw	
Total primary and secondary power	1,014,000 continuous kw	
Estimated unit cost of power per continuous kw		$444

Multiple-use system

Primary power	971,000 continuous kw	
Secondary power—229,000 kw Equivalent potential power (3/8 x 229,000)	86,000 continuous kw	
Total primary and secondary power	1,057,000 continuous kw	
Estimated alternative cost of power (1,057,000 x $444)		$469,308,000
Add cost of additional units		
2 Units at Pickwick Landing		$ 7,900,000
4 Units at Wilson		5,985,000
3 Units at Wheeler		7,255,000
3 Units at Cherokee		5,064,000
2 Units at Douglas		4,775,000
1 Unit at Hiwassee		5,154,000
1 Unit at Chatuge		2,330,000
1 Unit at Nottely		2,581,000
1 Unit at Fontana		3,470,000
Alternate justifiable expenditure for power		$513,822,000

Allocation

The allocation of common costs and resultant system costs after completion of current work in progress, as computed by the alternative-justifiable-expenditure method, is shown in table 77 (table X in report as submitted to the President).

After consideration of all factors, including the guidance provided by table 77 (table X in report as submitted to the President), it is the judgment of the Investment Allocation Committee that the percentages of common costs allocated to the several purposes should be those shown in table 78 (table XI in report as submitted to the President). The resulting allocation of investment in plant in service June 30, 1953, as shown in that table, is aproximately $924.5 million to power, $182.4 million to flood control, and $156.6 million to navigation.

REFERENCES

Official TVA allocation reports

Reports on the Allocation of the System Investment as Provided by Section 14 of the Tennessee Valley Authority Act.

June 9, 1938 — Report on the Investment in Wilson, Norris and Wheeler Projects. 46p. (Also issued as House Document No. 709, 75th Congress, 3d session.)

September 27, 1939 — Report on Investment in Wilson, Norris, Wheeler and Pickwick Landing Projects. 4p. (also Notes, 7p.)

November 18, 1940 — Report on Investment in Wilson, Norris, Wheeler, Pickwick Landing, Guntersville, Chickamauga, and Hiwassee Dams. 4p. (also Notes, 9p.)

December 12, 1942 — Report on Investment in Wilson, Norris, Wheeler, Pickwick Landing, Guntersville, Chickamauga, Hiwassee, Nottely, Watts Bar Hydro, Watts Bar Steam,

Chatuge, and Cherokee Projects . . . as of June 30, 1942. 4p. (also Notes, 12p.)

October 5, 1943 — Allocation Report . . . as of June 30, 1943. 5p.

October 7, 1944 — Allocation of Cost Report as of June 30, 1944. 5p.

November 13, 1945 — Report on Allocation of Costs as of June 30, 1945. 4p. (also Notes, 18p.)

November 21, 1950 — Report on Allocation of Costs as of June 30, 1950. 5p. (also Notes, 18p.)

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TABLE 77 *(Table X in report as submitted to the President).—Allocation of estimated system costs upon completion of work in progress.*

Purpose	Calculation of percentages for guidance in distribution of common costs: Alternative justifiable expenditures (Tables VI, VIII, and IX)	Direct investments (Table V)	Remaining alternative justifiable expenditures: Amount	Percent	Allocation of common costs	Total estimated investment multiple-purpose
Navigation	$ 231,818,000	$ 46,168,032	$185,649,968	27.6	$113,760,303	$159,928,335
Flood control	260,458,000	55,439,000	205,019,000	30.4	125,301,203	180,740,203
Power	513,822,000	230,779,799	283,042,201	42.0	173,113,505	403,893,304
Total	$1,006,098,000	$332,386,831	$673,711,169	100.0	$412,175,011	$744,561,842

TABLE 78 *(Table XI in report as submitted to the President).—Recommended allocation—investment in plant in service as of June 30, 1953.*

Purpose	Direct investments (Table IV)	Allocation of common costs: Percent	Amount	Single-purpose projects and other electric plant (Table II)	Total system investment: Amount	Percent
Navigation	$ 45,934,051	27.0	$110,646,189	—	$ 156,580,240	12.4
Flood control	55,367,000	31.0	127,038,217	—	182,405,217	14.4
Power	207,649,707	42.0	172,116,294	$544,747,489	924,513,490	73.2
Total	$308,950,758	100.0	$409,800,700	$544,747,489	$1,263,498,947	100.0

INDEX