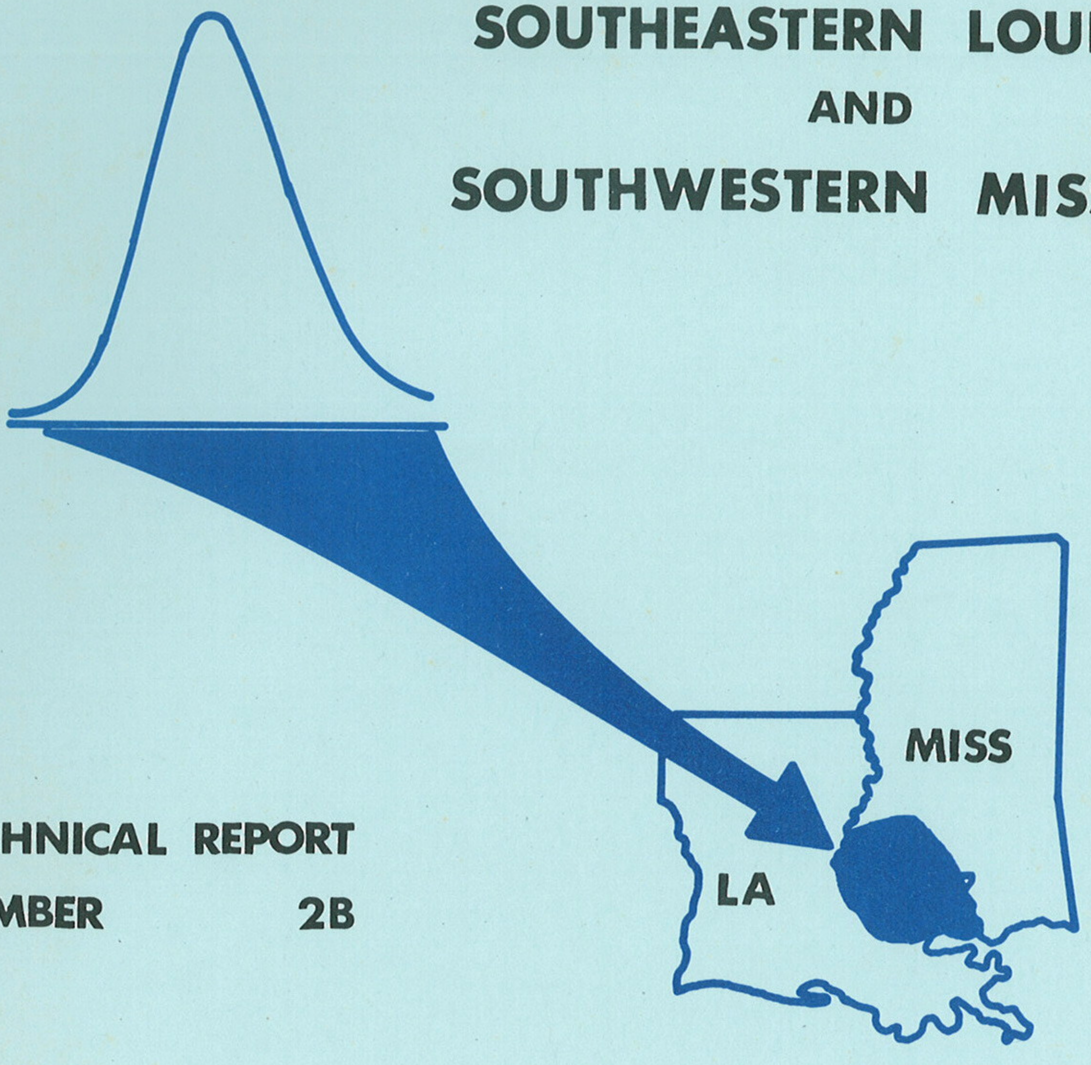
UNIT HYDROGRAPHS

FOR SOUTHEASTERN LOUISIANA AND SOUTHWESTERN MISSISSIPPI



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STATE OF LOUISIANA
DEPARTMENT OF PUBLIC WORKS

TECHNICAL REPORT NO. 2b

UNIT HYDROGRAPHS FOR SOUTHEASTERN LOUISIANA AND SOUTHWESTERN MISSISSIPPI

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PREFACE

In 1962, the Louisiana Department of Public Works and the U.S. Geological Survey agreed, as part of their cooperative program, to investigate and develop methods which could be used to reproduce or synthesize storm hydrographs of specific storms from basin characteristics and rainfall records. The original agreement was for southeast Louisiana, an area known locally as the "Florida Parishes". After this investigation started, it was found that certain streamgaging stations in southwestern Mississippi would greatly benefit the overall results; therefore, an area of about 4,000 square miles in southwestern Mississippi was also included. A similar study is in progress for an area of about 9,000 square miles in southwest Louisiana.

The project is divided into three basic phases: (1) rainfall-runoff relations, (2) unit-hydrographs, and (3) magnitude and frequency of storm runoff. Separate reports covering each phase will be published as a series of technical reports, as follows:

Technical Report No. 2a - Rainfall-Runoff Relations for Southeastern Louisiana and Southwestern Mississippi

No. 2b - Unit Hydrographs for Southeastern Louisiana and Southwestern Mississippi

No. 2c - Rainfall-Runoff Relations for Southwestern Louisiana

No. 2d - Unit Hydrographs for Southwestern Louisiana

No. 2e - Magnitude and Frequency of Storm Runoff in Southwestern Louisiana, Southeastern Louisiana, and Southwestern Mississippi.

One phase of the project has been published in U.S. Geological Survey Professional Paper 501-D. This paper, "Magnitude and Frequency of Storm Runoff in Southeastern Louisiana and Southwestern Mississippi", by V. B. Sauer, will be incorporated into Technical Report No. 2e. The five reports listed above will constitute a set which can be used to derive a storm hydrograph from rainfall records and basin characteristics in the area described.

CONTENTS

CONTENTS – Continued

ILLUSTRATIONS

ILLUSTRATIONS - Continued

TABLES

UNIT HYDROGRAPHS FOR SOUTHEASTERN LOUISIANA AND SOUTHWESTERN MISSISSIPPI

by

V. B. Sauer

ABSTRACT

Unit hydrographs and base-flow recession curves were developed from flood records for 17 gaging stations in the 7,500 square mile study area. These data, which are applicable at the gaging sites, can be used to estimate complete flood hydrographs on the basis of rainfall excess. The 17 unit hydrographs for the gaging stations were reduced to one dimensionless hydrograph dependent on the basin lag time and the hydrograph volume of discharge. Unit hydrographs can be developed for any ungaged stream in the study area using the dimensionless hydrograph, drainage area and lag time.

The "volume" (sum of discharges at hydrograph intervals) is a function of the drainage area (A), and can be expressed as,

$$\Sigma Q = \frac{645.3A}{\Delta t}$$

where Δt is a computation interval.

The relation of lag time (T_L) is expressed to the basin mean length (L_{ca}) as follows:

$$T_L = KL_{ca}^{0.8},$$

where K is a constant for each of three subareas.

Lag time can also be computed on the basis of drainage area in the formula,

$$T_L = bA^{0.5}$$

where b is a constant for each of the three subareas. In the application of either formula, an adjustment for duration is required.

The standard error of estimate of T_L using the basin-mean-length formula was about 10 percent, and based on the drainage area formula, was about 19 percent. The accuracy of the unit hydrograph depends to a large extent on the accuracy of T_L.

Base-flow recession data for the 17 gaging stations were combined into a family of curves suitable for estimating base-flow recessions at ungaged sites from drainage area and rainfall excess. The errors in these curves become insignificant when considered in light of the whole flood hydrograph.

INTRODUCTION

Much work has been published on the theory and application of methods for computing storm runoff from basin characteristics and rainfall records. Most of the reports that provide for the application of such methods are limited to a specific site or geographic area. This is generally necessary because the controlling factors in such analyses vary considerably from one locale to another. In fact, some factors are so complicated in nature that it would not be practical to evaluate them for most applications. In this respect, this report is limited in application to the areas defined. Data for 17 sites for which streamflow records are available have been analyzed and are presented in this report as specific site data. These site data have also been regionalized, and methods are presented for estimating the parameters that are needed to synthesize flood hydrographs at ungaged sites.

The unit hydrograph is a hydrologic tool that can be used to estimate the hydrographs of either actual or hypothetical floods from rainfall excess. It is useful for flood predictions, design of waterway structures and channels, estimation of missing streamflow records, and extension of flood records on the basis of long term rainfall records. The purpose of this report is to develop this tool for practical application to streams in the study area so that problems of this type can be more easily solved. The finer details of such items as the derivation of a unit-hydrograph from station data or the regionalization of site data, are discussed in general terms with only enough detail for the user to understand the application. If further explanation is desired, the references in "Selected References" should be consulted. The reference, in particular, that was used extensively in this study is "Unit Hydrographs in Illinois" by William D. Mitchell (1948).

The study area (figure 1) includes what is known as the "Florida Parishes" in southeastern Louisiana and about eight counties to the north of the Florida Parishes in southwestern Mississippi. It is bounded on the east by the Pearl River and on the west by the Mississippi River. Topography of the 7,500 square mile area is varied, ranging from rolling hills to flat, swampy lands. Average annual rainfall ranges from about 56 inches in southwestern Mississippi to 66 inches in southeastern Louisiana. A more detailed description of the area can be found in Technical Report No. 2a, "Rainfall-Runoff Relations for Southeastern Louisiana and Southwestern Mississippi".

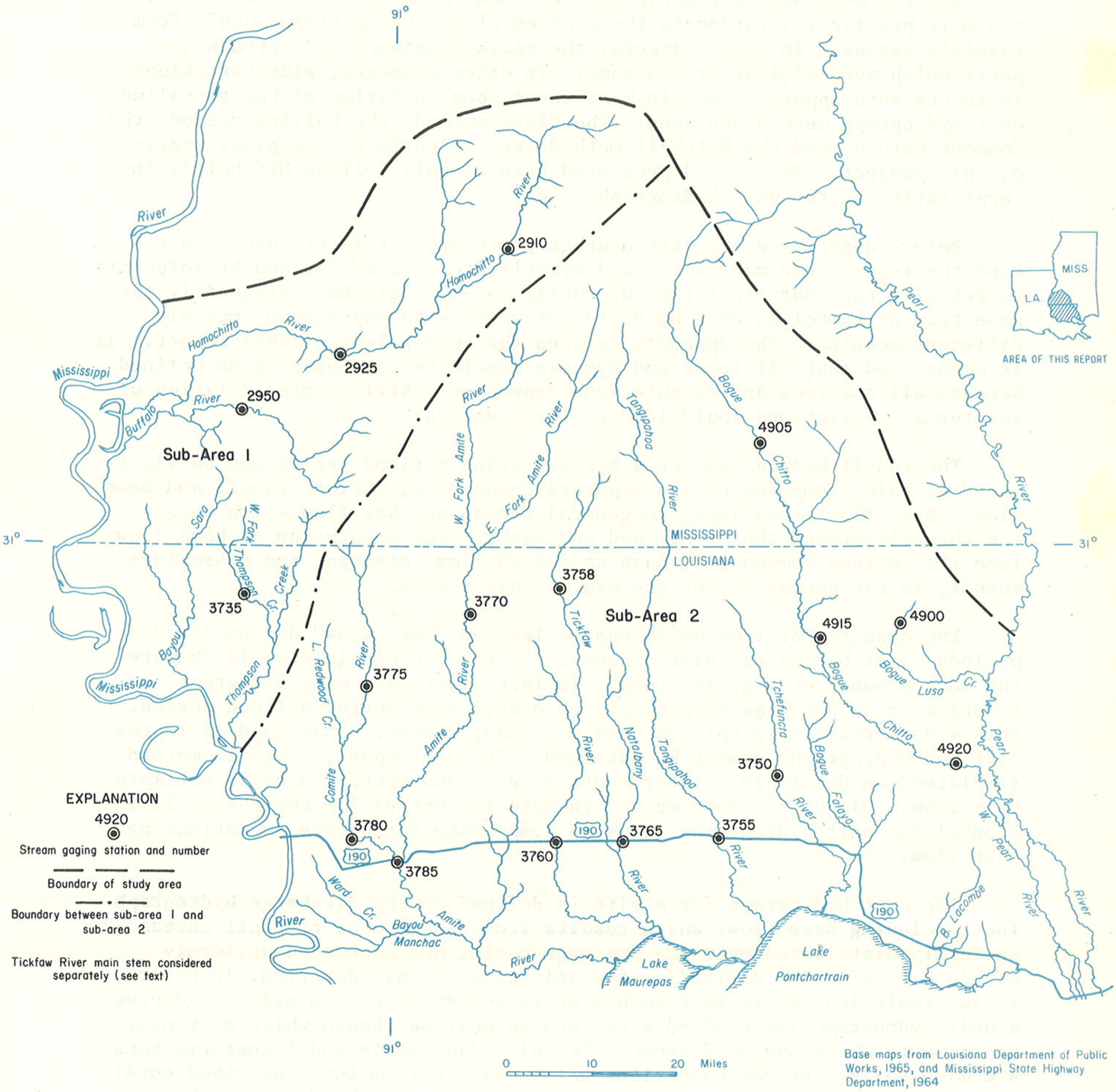


Figure 1.--Map of study area

UNIT-HYDROGRAPH THEORY

Several methods, or theories, were tried in an attempt to arrive at the most practical solution to the problem of computing storm runoff from rainfall excess. In some respects, the various methods had certain aspects which were similar or the same. In other respects, wide variations in theory were apparent, but in most cases some variation of the so-called unit hydrograph method was used. The Clark method, the Collins method, the Commons method, and the Mitchell method were tried in the original analyses of this project. The method presented here closely follows Mitchell's interpretation of the unit-hydrograph theory.

Before describing the unit-hydrograph theory, it is necessary to remind the reader that many terms and symbols are used by various hydrologists to describe the characteristics of runoff and hydrographs. Frequently the same term or symbol is used by different writers to express an entirely different meaning. The appendix defines the terminology of this report. It is emphasized that all terms and symbols should be used exactly as defined because all analyses and computations depend on a strict interpretation of the terms. Deviations could lead to large errors.

The runoff hydrograph for a stream during a flood period can be visualized as being composed of two separate components, surface runoff and base flow. Base flow is defined, in general terms, as that flow which enters the channel through the stream bed and banks. The separation of base flow into its various components, such as spring flow, seepage, and subsurface runoff, is not necessary for the use of this report.

The quantity of flow which can be labeled "base flow" during flood periods is at best a calculated guess. No two hydrologists would compute the same amount for a given storm. In fact, there are many different theories as to how base flow should be distributed during a flood period. This wide variation in opinion does not mean, however, that good estimates of flood hydrographs cannot be obtained. In this report, a single method (explained in detail in a later section) was consistently used to estimate base flow. Use of the same method in both the derivation and the application of the unit hydrographs tends to compensate for errors in estimating base flow.

The unit hydrograph for a site is defined as the discharge hydrograph (not including base flow) which results from one inch of rainfall excess uniformly distributed over the drainage basin, and generated uniformly within a time period defined as the unit time or unit duration. In nature, it is highly improbable that such a storm would occur. In order to derive a unit hydrograph for a gaged site, storms must be chosen which most nearly approximate the above conditions. Certain adjustments and transformations are made to account for conditions which deviate from the prescribed conditions. The details of deriving a unit hydrograph can be found in the reference material, particularly Mitchell (1948).

Figure 2 is a sketch showing a typical unit hydrograph (not including base flow) and its relation to rainfall excess. The various dimensions in this sketch are explained in this and other sections of this report. Definitions can also be found in the appendix.

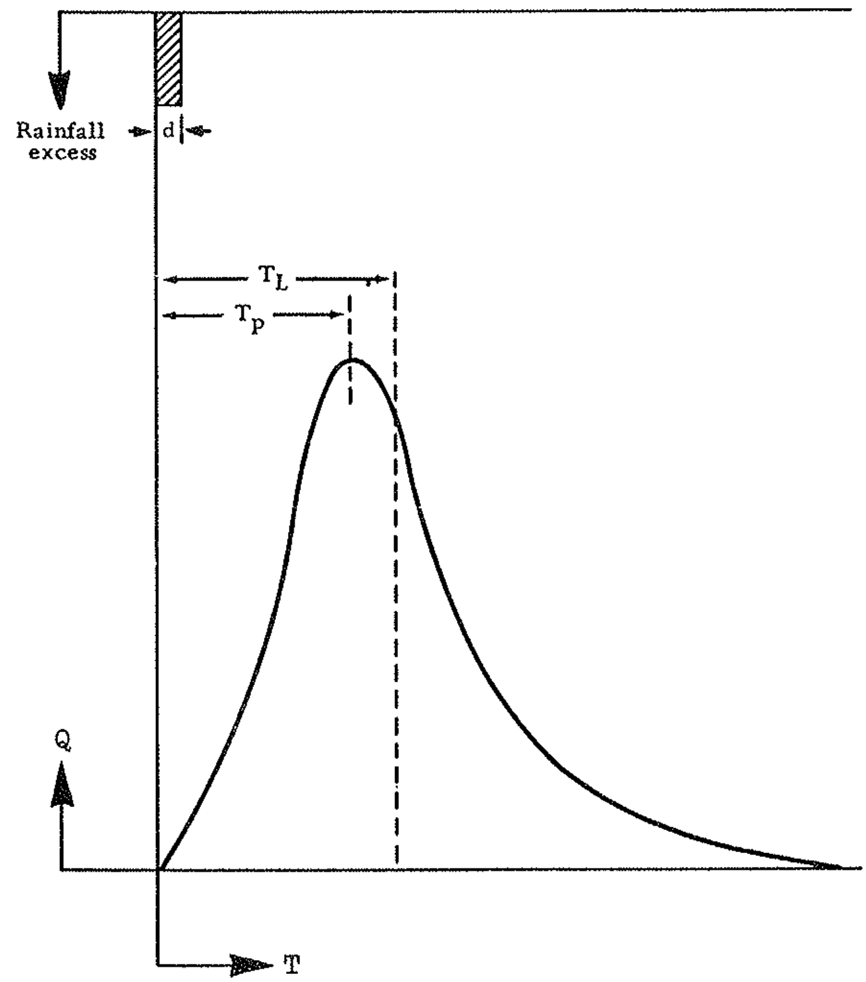


Figure 2.--Sketch of typical unit hydrograph.

The unit hydrograph for a site can be used to reproduce other hydrographs at the same site resulting from rainfalls of any amount provided that certain assumptions are met. These assumptions are derived from the definition above and are as follows:

(1) It is assumed that the rainfall excess of a particular storm can be determined with reasonable accuracy. Not only must the volume of rainfall excess be determined, but just as important, the time distribution must be known. Rainfall-runoff relations for southeastern Louisiana and southwestern Mississippi are contained in Technical Report No. 2a. The derivation, use, and accuracy of these relations are explained in detail in that report. It is expected that rainfall excess will be derived as explained in that report, although the basic principles of the unit-hydrograph theory do not depend on the manner in which rainfall excess is computed. Any method which gives reasonably accurate approximations of rainfall excess will do. It should be pointed out that the unit hydrograph is not a tool for computing rainfall excess, but only a method by which rainfall excess can be converted to a discharge hydrograph.

(2) It is assumed that the runoff-producing rainfall is distributed fairly uniformly over the basin. This assumption limits, to some extent, the maximum size of basins which can be used in such computations. For the basins encountered in the study area (all less than 1,500 square miles), it can generally be assumed that uniform distribution will occur for the large storms; however, the user should assure himself of uniform areal distribution for any storm to be computed because in some instances rainfall may be concentrated over one part of a basin, or the storm may move upstream or downstream, all of which tend to distort the hydrograph for that storm. Because there are always some nonuniformities and because rainfall excess is difficult to compute with accuracy, it cannot be expected that exact reproductions will be obtained. If it is desired to compute the flood hydrograph for an outstanding storm over one of the larger basins and it is known that this storm is not uniformly distributed, the basin can be subdivided into smaller basins, and the hydrographs computed for each. After this has been done, flood routing procedures can be used to combine the various sub-basin hydrographs at the desired location. The reference by Carter and Godfrey (1960) provides a suitable method of routing floods.

(3) For a given site, it is assumed that discharge ordinates at corresponding times of direct-runoff hydrographs resulting from different volumes of rainfall excess generated in unit time are in the same proportion as the volumes of rainfall excess. For example, if the peak discharge for 1 inch of runoff (occurring in unit duration) is 1,000 cfs, then the peak discharge for 2 inches of runoff (again occurring in unit duration) will be 2,000 cfs. Mitchell has demonstrated, in a project still in progress, that this assumption is true if the relation between channel storage and discharge is linear. He has also devised a method to determine if this relation is linear. Based on his preliminary methods, the unit hydrographs at streamflow sites presented in this report were

tested for discharge-storage linearity and found to be linear within reasonable limits. Consequently, the assumption of proportionality can be considered valid for the streams in the study area.

Once a unit hydrograph has been derived for a site, whether it is derived from station data or from synthetic methods, it can then be applied to a storm. If the duration of rainfall excess for a particular storm corresponds to the unit duration which was used to derive the unit hydrograph, the surface-runoff hydrograph can be computed for that storm by multiplying each ordinate of the unit hydrograph by the total rainfall excess, in inches. If, however, the rainfall excess occurs over a longer time period than the unit time, it must be subdivided into increments of rainfall excess for time units equal to the unit time. For instance, suppose rainfall excess occurred for 6 hours with hourly totals as follows:

Time interval	Rainfall excess, in inches
0600-0700	0.25
0700-0800	0.53
0800-0900	0.98
0900-1000	0.45
1000-1100	0.63
1100-1200	0.27

It is desired that a 2-hour unit hydrograph be used to generate the runoff hydrograph for this storm. It would be necessary then to combine successive hourly totals as 2-hour totals for computation purposes. The following distribution would result from this combination:

Time interval	Rainfall excess, in inches
0600-0800	0.78
0800-1000	1.43
1000-1200	0.90

Unit-hydrograph ordinates would be multiplied by each of these 2-hr rainfall excess totals to produce three distinct runoff hydrographs. These hydrographs would then be successively lagged, each by 2 hours, and summed to produce the total surface-runoff hydrograph. The surface-runoff hydrograph does not represent total flow. To complete the hydrograph, base flow, which is not accounted for by the unit hydrograph, must now be added to the ordinates of surface runoff. The procedure for estimating base flow is explained in a later section.

GAGING STATION DATA

Site data were computed at 17 gaging stations located in the study area. For each station, a unit hydrograph and base-flow recession were computed. Also computed are physical parameters of the basin, namely, basin size, length, and mean length. Time factors, T_p, and T_L, are computed from the given unit hydrograph data.

The following pages contain a tabulation of data for each gaging station analyzed for this report. The unit hydrograph data are given at time intervals equal to the unit duration best suited for each particular station. If other than the given unit duration is desired, then the unit hydrograph should be transformed to the desired duration of unit time. Details for such transformations are given in the reference material by Mitchell (1948).

The base-flow recession data are average for the year. No attempt was made to determine individual base-flow curves for the various seasons of the year because small variations of base flow will not cause significant errors in the final hydrograph. The upper limits of base flow, as shown, are estimated on the basis of extrapolation of the known recessions. The time interval between successive points was chosen only as a convenient plotting interval. Other time intervals may be used by simply interpolating between the given points. If the data must be extrapolated above or below the limits shown, it is recommended that the given data be plotted on semi-log plotting paper and extrapolated by straight line extension. The numbers in parenthesis represent inches of storm runoff and should be used as mergence points for storms of the indicated size. For instance, to add base flow to a storm with 2 inches of rainfall excess on Bogue Lusa Creek near Franklinton, the time when surface runoff ends would be the time to merge the base-flow recession at a discharge of 83 cfs. The base-flow recession is then projected back from this point at the rate shown in the table. The complete procedure for estimating base flow during a flood period is given in the section, "Base-flow estimates".

2-4900 Bogue Lusa Creek near Franklinton, La.

Location.--Lat 30°52'05", long 90°00'10", in NE1/4NW1/4 sec. 39, T. 2 S., R. 12 E., St. Helena meridian, near right bank at downstream side of bridge on State Highway 10, three-quarters of a mile upstream from Witches Creek, and 9 miles east of Franklinton.

Drainage area (A).--12.1 square miles.
Basin length (L).--5.4 miles.
Basin mean length (L_{ca}).--2.7 miles.
Time-to-peak (T_p).--2.5 hours.
Adjusted lag time (T_L).--6.1 hours.

Unit hydrograph data, cfs (d = Δt = 1 hour)

322	430	120
840	360	93
1400	300	70
1020	250	50
820	205	30
640	170	20
520	140	10

Base flow recession data, cfs (Δt = 4 hours)

200	110 (3)	64
180	100	59
170	91	53
160	83 (2)	49 (1)
140 (4)	76	45
130	70	41
120		

2-4905 Bogue Chitto near Tylertown, Miss.

Location.--Lat 31°11', long 90°17', in SE1/4 sec. 34, T. 3 N., R. 9 E., Washington meridian, near right bank on downstream side of bridge on U.S. Highway 98, a quarter of a mile upstream from Fernwood, Columbia and Gulf Railroad Co. bridge, a quarter of a mile upstream from Bars Branch, 2 1/4 miles downstream from Topisaw Creek, and 9 miles northwest of Tylertown.

Drainage area (A).--502 square miles.
Basin length (L).--45.0 miles.
Basin mean length (L_{ca}).--22.7 miles.
Time-to-peak (T_p).--27 hours.
Adjusted lag time (T_L).--41.8 hours.

Unit hydrograph data, cfs (d = Δt = 6 hours)

864	4700	648
1780	3560	432
3620	2750	270
6260	2160	216
8480	1730	162
7720	1300	108
6260	918	52

Base flow recession data, cfs (Δt = 6 hours)

1830	1420	1100	850
1790	1380	1070 (3)	830
1730	1340	1040	800
1690	1300	1010	780 (1)
1630	1260	980	760
1600	1220 (4)	950	740
1550	1200	920 (2)	720
1500	1160	900	700
1460	1130	880	

2-4915 Bogue Chitto at Franklinton, La.

Location.--Lat 30°50'35", long 90°09'45", in SE1/4SE1/4 sec. 26, T. 2 S., R. 10 E., on right bank just downstream from bridge on State Highway 10, three-quarters of a mile west of Franklinton and 3 1/2 miles upstream from Lawrence Creek.

Drainage area (A).--985 square miles.
Basin length (L).--65.3 miles.
Basin mean length (L_{ca}).--34.9 miles.
Time-to-peak (T_p).--33 hours.
Adjusted lag time (T_L).--47.5 hours.

Unit hydrograph data, cfs (d = Δt = 6 hour)

1140	8210	1820
2440	6680	1430
5510	5190	1060
9960	4130	740
12700	3390	480
13700	2810	210
12200	2280	50
9860		

Base flow recession data, cfs (Δt =12 hours)

6200	3300 (4)	1700 (1)
5700	3000	1600
5200	2700 (3)	1400
4700	2500	1300
4300	2300 (2)	1200
3900	2100	1100
3600	1900	1000

2-4920 Bogue Chitto near Bush, La.

Location.--Lat 30°37'45", long 89°53'50", in T. 5 S., R. 13 E., near Center of span on downstream side of bridge on State Highway 21, 0.2 mile downstream from Gulf, Mobile, and Ohio Railroad bridge, and 1.4 miles north of Bush.

Drainage area (A).--1210 square miles.
Basin length (L).--92.4 miles.
Basin mean length (L_{ca}).--53.2 miles.
Time-to-peak (T_p).--57 hours.
Adjusted lag time (T_L).--65.8 hours.

Unit hydrograph data, cfs (d = Δt = 6 hour)

52	15800	1630
104	14600	1170
365	12800	782
1560	10600	521
3840	7930	325
6120	5860	195
8720	4300	130
12200	3060	66
15200	2210	

Base flow recession data, cfs (Δt =12 hours)

9000	4400	2200
8000	4000	2000
7200	3500 (4)	1700 (1)
6400	3100 (3)	1500
5600	2800	1400
5000	2500 (2)	1200

7-2910 Homochitto River at Eddiceton, Miss.

Location.--Lat 31°30', long 90°47', near center of sec. 11, T. 6 N., R. 4 E., Washington meridian, on left bank at upstream side of Mississippi Central Railroad Co. bridge, 900 ft. downstream from bridge on U.S. Highway 84, 0.4 mile upstream from McCall Creek, and three-quarters of a mile east of Eddiceton.

Drainage area (A).--180 square miles.
Basin length (L).--30.0 miles.
Basin mean length (L_{ca}).--16.1 miles.
Time-to-peak (T_p).--5 hours.
Adjusted lag time (T_L).--9.0 hours.

Unit hydrograph data, cfs (d = Δt = 2 hours)

1920	2620
10800	1920
13600	1390
9350	928
6500	580
4590	347
3430	115

Base flow recession data, cfs (Δt = 4 hours)

1500	990	660
1400	920 (4)	610
1300	860 (3)	570
1220	800 (2)	530
1140	750 (1)	500
1060	700	460

7-2925 Homochitto River at Rosetta, Miss.

Location.--Lat 31°19'20", long 91°06'20", in sec. 12, T. 4 N., R. 1 E., Washington meridian, on downstream side of bridge on State Highway 33 at Rosetta, 800 ft. downstream from Illinois Central Railroad Co. bridge, 1 mile downstream from Foster Creek, and 5 miles upstream from Dry Creek.

	Unit hydrograph data, cfs (d = Δt = 4 hours)			Base flow recession data, cfs (Δt = 8 hours)		
Drainage area (A).--750 square miles.	1330	6780	1330	3400	2310	1580 (2)
Basin length (L).--65.0 miles.	7020	4840	969	3200	2200 (4)	1490
Basin mean length (L_{ca}).--36.1 miles.	33900	3750	605	3050	2080	1410
Time-to-peak (T_p).--10 hours.	25900	3020	369	2900	1980	1340 (1)
Adjusted lag time (T_L).--20.3 hours.	16200	2300	242	2730	1850 (3)	1260
	10400	1940	121	2600	1750	1200
				2440	1650	1140

7-2950 Buffalo River near Woodville, Miss.

Location.--Lat 31°13'35", long 91°17'45", in SW1/4 sec. 21, T. 3 N., R. 2 W., Washington meridian, near center of span on downstream side of bridge on U.S. Highway 61, 1 1/2 miles downstream from Fords Creek, 2 3/4 miles west of Wilkinson, and 8 1/2 miles north of Woodville.

	Unit hydrograph data, cfs (d = Δt = 2 hours)			Base flow recession data, cfs (Δt = 6 hours)		
Drainage area (A).--182 square miles.	705	2580	411	1440	790	430
Basin length (L).--27.5 miles.	3170	1760	294	1370	750 (3)	410
Basin mean length (L_{ca}).--14.3 miles.	9280	1230	235	1300	710	390
Time-to-peak (T_p).--7 hours.	14200	940	176	1240	680	370
Adjusted lag time (T_L).--10.8 hours.	11600	705	117	1180	650	360
	6520	529	59	1120	610	340
	3700	470		1060	590	320
				1010 (4)	560	300 (1)
				960	530 (2)	290
				920	500	280
				870	480	270
				830	450	

7-3735 West Fork Thompson Creek near Wakefield, La.

Location.--Lat 30°55'20", long 91°17'35", in lot 43, T. 1 S., R. 2 W., St. Helena meridian, near right bank on downstream side of bridge on State Highway 421, 3 1/2 miles northeast of Wakefield, and 4 1/2 miles upstream from Middle Fork Thompson Creek.

	Unit hydrograph data, cfs (d = Δt = 1 hour)			Base flow recession data, cfs (Δt = 4 hours)		
Drainage area (A).--35.3 square miles.	70		860	340	200	118
Basin length (L).--15.1 miles.	1210		560	320	190	110
Basin mean length (L_{ca}).--7.6 miles.	3460		350	300	180	104
Time-to-peak (T_p).--3.5 hours.	5150		210	280	170	98 (1)
Adjusted lag time (T_L).--5.3 hours.	4580		100	270	160	93
	3070		40	250 (4)	150 (2)	88
	1850		20	240	140	83
	1240		10	220	132	78
				210 (3)	125	74

7-3750 Tchefuncta River near Folsom, La.

Location.--Lat. 30°36'55", long 90°14'55", on line between SE1/4NE1/4 and SW1/4NE1/4 sec. 13, T. 5 S., R. 9 E., St. Helena meridian, near center of span on downstream side of bridge on State Highway 40, 1.2 miles upstream from Bull Branch, and 3.6 miles southwest of Folsom.

	Unit hydrograph data, cfs (d = Δt = 3 hours)			Base flow recession data, cfs (Δt =12 hours)		
Drainage area (A).--103 square miles.	620	886	175	820	300 (3)	130
Basin length (L).--21.2 miles.	1610	674	144	700	260	110
Basin mean length (L_{ca}).--11.0 miles.	4050	530	93	590	220 (2)	94
Time-to-peak (T_p).--12 hours.	4110	425	64	500	180	80
Adjusted lag time (T_L).--17.3 hours.	2970	343	49	420 (4)	160 (1)	68
	2110	286	24	360		
	1570	233	7			
	1180					

7-3755 Tangipahoa River at Robert, La.

Location.--Lat 30°30'23', long 90°21'42", in lot 39, T. 6 S., R. 8 E., St. Helena meridian, on right bank just downstream from bridge on U.S. Highway 190, 1 mile west of Robert, 2 miles downstream from Chappepeela Creek, and 6 miles east of Hammond.

	Unit hydrograph data, cfs (d = Δt = 6 hours)			Base flow recession data, cfs (Δt =12 hours)		
Drainage area (A).--646 square miles.	834	7360	834	6600	3200 (4)	1540
Basin length (L).--66.0 miles.	1810	6610	625	6000	2900	1410
Basin mean length (L_{ca}).--33.9 miles.	2710	5140	486	5500	2700	1290
Time-to-peak (T_p).--45 hours.	3680	4100	347	5000	2400 (3)	1170
Adjusted lag time (T_L).--52.0 hours.	5140	2780	208	4600	2200	1070
	6530	1880	139	4200	2000	980 (1)
	7710	1460	69	3800	1850	900
	7850	1180		3500	1700 (2)	820

7-3758 Tickfaw River at Liverpool, La.

Location.--Lat 30°55'47", long 90°40'41", on line between sec. 46 and 47, T. 1 S., R. 5 E., St. Helena meridian, near left bank on downstream side of bridge on State Highway 38, 0.5 mile east of intersection of State Highway 38 and 43, 0.5 mile upstream from Cotton Patch Branch, and 1 mile north of Liverpool.

	Unit hydrograph data, cfs (d = Δt = 3 hours)			Base flow recession data, cfs (Δt = 6 hours)	
Drainage area (A).--89.7 square miles.	116	2160	502	1080	230 (3)
Basin length (L).--16.9 miles.	289	1780	405	890	190
Basin mean length (L_{ca}).--8.6 miles.	482	1520	289	730	160 (2)
Time-to-peak (T_p).--22.5 hours.	675	1270	212	610	130
Adjusted lag time (T_L).--30.2 hours.	791	1080	154	500	110 (1)
	984	907	96	410	90
	1620	753	58	340	74
	2510	617	19	280 (4)	61

7-3760 Tickfaw River at Holden, La.

Location.--Lat 30°30'13", long 90°40'38", in sec. 26, T. 6 S., R. 5 E., St. Helena meridian, near left bank on downstream side of bridge on U.S. Highway 190, 1/2 mile west of Holden, and 5.1 miles upstream from Big Branch.

	Unit hydrograph data, cfs (d = Δt = 6 hours)			Base flow recession data, cfs (Δt =12 hours)		
Drainage area (A).--247 square miles.	30	1490	1010	1000	530 (4)	320
Basin length (L).--49.3 miles.	110	1650	800	910	490	290
Basin mean length (L_{ca}).--29.1 miles.	210	1780	580	840	450 (3)	270 (1)
Time-to-peak (T_p).--81 hours.	450	1810	420	760	410	240
Adjusted lag time (T_L).--86.1 hours.	660	1780	320	700	380	220
	820	1700	240	640	350 (2)	200
	980	1620	160	590		
	1090	1540	110			
	1200	1380	50			
	1330	1220	30			

7-3765 Natalbany River at Baptist, La.

Location.--Lat 30°31'15", long 90°32'45", in NE1/4NW1/4 sec. 30, T. 6 S., R. 7 E., St. Helena meridian, near right bank on downstream side of bridge on U.S. Highway 190, 0.7 mile downstream from Still Branch, and 0.7 mile west of Baptist.

	Unit hydrograph data, cfs (d = Δt = 3 hours)			Base flow recession data, cfs (Δt = 6 hours)		
Drainage area (A).--79.5 square miles.	40	980	240	1030	430	180
Basin length (L).--30.9 miles	490	820	200	890	380	160
Basin mean length (L_{ca}).--16.8 miles.	920	690	160	770	320 (4)	135 (1)
Time-to-peak (T_p).--16.5 hours.	1370	600	120	670	280 (3)	118
Adjusted lag time (T_L).--26.8 hours.	1670	520	80	580	240	102
	1860	450	60	500	210 (2)	
	1840	400	40			
	1600	340	20			
	1260	290				

7-3770 Amite River near Darlington, La.

Location.--Lat 30°53'20", long 90°50'40", in lot 72, T. 2 S., R. 4 E., St. Helena meridian, on left bank just downstream from bridge on State Highway 10, 1.5 miles upstream from Collins Creek, and 4.0 miles west of Darlington. Prior to July 30, 1963 at former channel 700 ft. to the left

	Unit hydrograph data, cfs (d = Δt = 4 hours)			Base flow recession data, cfs (Δt =12 hours)	
Drainage area (A).--580 square miles.	850	9540	1260	4000	1900 (3)
Basin length (L).--41.6 miles.	1920	7200	940	3600	1700
Basin mean length (L_{ca}).--22.5 miles.	3320	5470	700	3200	1530 (2)
Time-to-peak (T_p).--26 hours.	5520	4210	520	2900	1380 (1)
Adjusted lag time (T_L).--33.5 hours.	8420	3320	370	2600	1240
	10700	2620	230	2350	1120
	11500	2060	140	2100 (4)	1010
	11100	1640			

7-3775 Comite River near Olive Branch, La.

Location.--Lat 30°45'21", long 91°02'38", in lot 41, T. 3 S., R. 2 E., St. Helena meridian, near center of span on downstream side of bridge on State Highway 67, 1800 ft. downstream from Knighton Bayou, and 1.3 miles northeast of Olive Branch. Prior to Feb. 4, 1964, at site 1,400 ft. upstream.

	Unit hydrograph data, cfs (d = Δt = 4 hours)			Base flow recession data, cfs (Δt = 8 hours)		
Drainage area (A).--145 square miles.	1300	2190	240	1220	530 (4)	230
Basin length (L).--23.2 miles.	2960	1870	120	1050	460	200 (1)
Basin mean length (L_{ca}).--12.8 miles.	4090	1390	100	920	400 (3)	170
Time-to-peak (T_p).--10 hours.	3460	770	50	800	350	150
Adjusted lag time (T_L).--20.5 hours.	2640	430	20	700	300 (2)	130
	2400			600	260	

7-3780 Comite River near Comite, La.

Location.--Lat 30° 30'45", long 91°04'25", in NW1/4 sec. 24, T. 6 S., R. 1 E., St. Helena meridian, near left bank on downstream side of bridge on State Highway 946, 1/2 mile downstream from Blackwater Bayou, and 2.6 miles west of Comite.

Drainage area (A).--284 square miles.
Basin length (L).--41.6 miles.
Basin mean length (L_{ca}).--21.3 miles.
Time-to-peak (Tp).--30 hours.
Adjusted lag time (T_L).--37.3 hours.

Unit hydrograph data, cfs (d = Δt = 4 hours)

160	3820	1030
1050	3660	690
2240	3390	440
3020	3020	250
3410	2610	140
3640	2240	70
3780	1810	50
3850	1420	20

Base flow recession data, cfs (Δt = 8 hours)

1800	920	470
1600	820	420 (1)
1430	740 (4)	380
1280	660 (3)	340
1150	590	300
1030	530 (2)	270

7-3785 Amite River near Denham Springs, La.

Location.--Lat 30°27'50", long 90°59'25", in lot 2, T. 7 S., R. 2 E., St. Helena meridian, on left bank, between two adjacent bridges on U.S. Highway 190, 1000 ft. downstream from Comite River, 3 miles southwest of town of Denham Springs, and 15 miles east of Baton Rouge.

Drainage area (A).--1280 square miles.
Basin length (L).--79.5 miles.
Basin mean length (L_{ca}).--42.7 miles.
Time-to-peak (Tp).--51 hours.
Adjusted lag time (T_L).--60.1 hours.

Unit hydrograph data, cfs (d = Δt = 6 hours)

1100	12300	3920
2890	12200	2890
4680	11300	2070
6270	9850	1380
7710	8470	900
9090	7230	550
10400	5990	250
11400	4890	

Base flow recession data, cfs (Δt =12 hours)

5900	2400 (4)
5200	2100 (3)
4500	1810 (2)
4000	1600
3500	1400 (1)
3000	1220
2700	1060

UNIT HYDROGRAPHS FOR UNGAGED SITES

The preceding section presents various data necessary to apply unit hydrographs at gaged sites. All of these data were derived from actual streamflow records. It can naturally be expected that this is probably the most accurate means of deriving unit hydrograph data; however, in many applications it will be unlikely that gaging stations records are available at the site of application. Of course, if the site is near a streamflow station on the same stream, the known records can be transferred to the desired site by flood routing procedures, but many times it will be necessary to derive unit hydrographs by synthetic methods.

Unit hydrographs for different sites appear, at first glance, to have quite different shapes, and one might doubt that a group of unit hydrographs such as those for which data are presented in the preceding section, could be combined into a single hydrograph representing all. However, certain mathematical manipulations can be used to change the unit hydrograph into a dimensionless form. Dimensionless unit hydrographs are similar in shape and magnitude and can be averaged into a single unit hydrograph which can be used to reproduce synthetic unit hydrographs at ungaged sites. The method of reducing a unit hydrograph to dimensionless form involves, first, a transformation of the time scale by dividing each unit of time by the adjusted lag time of the unit hydrograph. Second, ordinates of discharge are determined at equal intervals of the transformed time scale and these ordinates of discharge are reduced to dimensionless values by dividing each by the summation of all. A group of unit hydrographs reduced to dimensionless form in this manner, can be averaged into one dimensionless hydrograph which will be representative of all. Such a procedure is referred to as regionalization. A more detailed explanation of regionalization is given by Mitchell (1948).

The station data in the preceding section were regionalized for the study area and then tabulated for convenient use. This section presents methods for estimating a suitable unit duration, lag time, and synthetic unit hydrograph. All tables and formulas were tested against the actual station data and found to give reasonable results (See "Accuracy and Limitations").

Selection of Unit Duration

The unit duration, d, by definition, is the time during which rainfall excess occurs to produce a unit hydrograph. Unit duration should be selected so that an optimum number of points are computed to define the unit hydrograph. Selection of a unit duration that is too small will result in excessive computations. This will not affect accuracy but will be laborious and time consuming. Selection of a unit duration that is too large will result in insufficient definition of the unit hydrograph and could lead to large errors. It has been found by experience that the optimum value of d can be chosen on the basis of lag time. Estimation of lag time, as explained in the next section, requires an adjustment based on unit duration, d. This interrelation of adjusted lag time and unit duration presents the problem that one must be known before the other can be computed. This, however, is not a serious problem because adjusted lag time needs only to be known within fairly broad limits to compute d. A rough estimate of lag time can be made from one of the formulas in the following section and used to enter the following table. This will usually provide a good selection of unit duration.

Adjusted lag time, T_L, in hours	Unit duration, d, in hours
less than 8	1
8 - 14	2
15 - 29	3
30 - 44	4
more than 44	6

Estimation of Lag Time

Lag time is defined as the time measured from center of mass of rainfall excess to the center of mass of resulting runoff. It has been demonstrated by Mitchell and others that lag time at a particular site will not vary from storm to storm provided that certain of the basic assumptions are met. The lag time computed for each station analyzed for this report was based on the final unit hydrograph, which is considered to be the hydrograph resulting from idealized conditions. Lag time was then correlated with various basin parameters to obtain methods for estimating lag time at ungaged sites.

In the application of this report, lag time has been adjusted slightly to facilitate easier usage. This adjustment is simply the addition of one-half the unit duration, or $\frac{d}{2}$, so that all computations will begin at the beginning of rainfall excess. By making this adjustment to lag time, no further adjustments are necessary for plotting the final hydrograph. The adjustment should not be overlooked because all other computations are based on the adjusted lag time, designated throughout the report as T_L.

It is stressed here that every means should be considered to obtain a good estimate of lag time. The accuracy of the synthetic hydrograph depends, to a large extent, upon the accuracy with which lag time is determined.

<u>Lag time estimated from mean length of basin</u>.--It was found that the best estimate of lag time could be made from the mean length, L_{ca}, of the basin. Mean length of the basin is computed as follows:

(1) On a scale map of the basin that shows the stream pattern, subdivide the main channel into equal lengths, preferably multiples of a mile, starting at the desired site. For most basins, it is recommended that at least 20 subreaches be used. All tributaries are subdivided in the same manner, using the same starting point and the

same interval as used for the main channel. Figure 3 illustrates the subdivided streams for a hypothetical basin. Connect all points equidistant from the starting point by a line which generally traverses the basin. All points along one of these lines are considered to be the same distance from the starting point, if the distance is measured along the stream channels.

(2) Planimeter each area between lines.

(3) Tabulate the data in a form similar to that in the table shown in figure 3. This permits a rapid calculation of the mean length as shown.

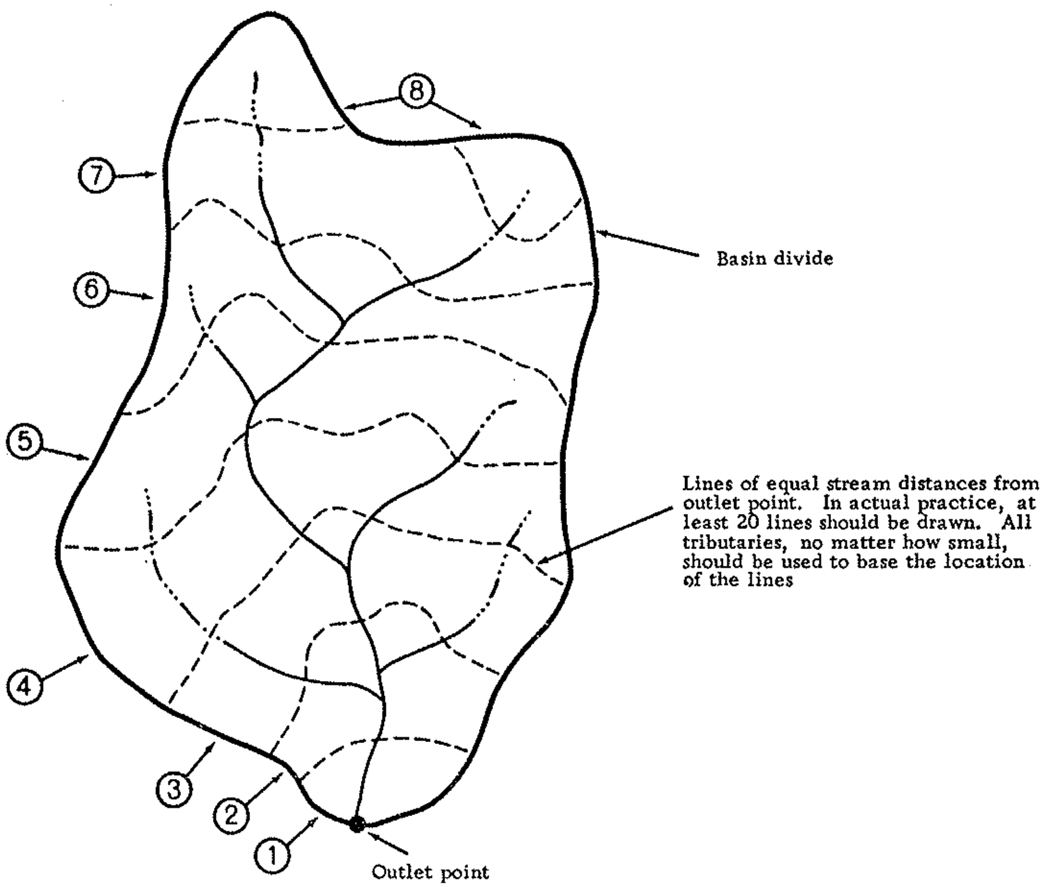
The lag time, when correlated with mean length for the gaging stations used in this analysis, indicated a definite division of the study area into sub-areas (fig. 1). It is evident, therefore, that other parameters have an effect on the lag time. Channel slope was investigated, but it did not improve the correlation, and it was concluded that the relatively flat slopes encountered throughout the study area do not differ enough to show any significant effect.

Another factor, channel storage, varies significantly within the study area and probably accounts for some of the variation in the lag relations. For instance, the Tickfaw River flood plain contains a braided low-water channel system with many interconnections. It is evident that channel storage is large for this stream throughout its length. This river is the only such stream encountered in the study, and lag time for it was found to be twice as large as for equivalent basins in the surrounding area. Conversely, the streams in sub-area 1 (see figure 1) indicated lag times considerably lower than those encountered in sub-area 2. Main channels of streams in sub-area 1 are generally deep and wide as compared to those of streams in sub-area 2, indicating a larger concentration of flow in the main channel. Lower overall roughness and more concentrated flow in this type of stream may account for the shorter lag times. Another factor which may be significant is channel meandering. It was noted that streams in sub-area 1 do not meander nearly as much as streams in sub-area 2. This affects the lengths of main stems because stream lengths as computed for this report followed the flood plain rather than the meandering low water channel. Formulas for computing adjusted lag time for the three described conditions are,

$$T_L = 1.0\, L_{ca}^{0.8} + \frac{d}{2} \text{ (for sub-area 1),} \qquad (1)$$

$$T_L = 2.8\, L_{ca}^{0.8} + \frac{d}{2} \text{ (for sub-area 2),} \qquad (2)$$

$$T_L = 5.6\, L_{ca}^{0.8} + \frac{d}{2} \text{ (Tickfaw River main stem)} \qquad (3)$$



Note: Areas and distances are estimated for this illustration.

Section	Area of section in square miles	Stream distance from outlet to midpoint of section, in miles	Product of Area X stream distance	Calculations
1	0.75	0.5	0.38	Total area = 22.86 sq. mi.
2	2.17	1.5	3.26	Σ Product = 95.81
3	3.77	2.5	9.42	$L_{ca} = \frac{95.81}{22.86} = 4.19$ miles
4	4.26	3.5	14.91	
5	4.02	4.5	18.09	
6	2.98	5.5	16.39	
7	3.31	6.5	21.52	
8	1.60	7.4	11.84	
Totals	22.86	---	95.81	

Figure 3.--Sketch of hypothetical basin illustrating computation of Lca

Based on these three formulas, the standard error of estimate of T_L at the 17 sites of this study was 10.2 percent.

Lag time estimated from drainage basin size.--The second method for estimating lag time is based on drainage area size and is not considered as accurate as the method using mean length of basin. Again, for the same reasons, the study area was divided into the same sub-areas as used for the mean-length correlations (see fig. 1). The corresponding formulas derived from these correlations are,

$$T_L = 0.7A^{.5} + \frac{d}{2} \text{ (for sub-area 1)}, \qquad (1a)$$

$$T_L = 1.7A^{.5} + \frac{d}{2} \text{ (for sub-area 2)}, \qquad (2a)$$

and

$$T_L = 4.2A^{.5} + \frac{d}{2} \text{ (for Tickfaw River main stem)}. \qquad (3a)$$

Based on these three formulas the standard error of estimate of T_L at the 17 sites of this study was 18.9 percent.

Lag time estimated from time-to-peak estimations.--A good correlation was found between time-to-peak and lag time. A single formula applies for the whole study area and excellent estimates of lag time can be made, provided that time-to-peak can be determined accurately for unit hydrograph conditions. At first, it might seem that the time-to-peak for one or two floods could be determined, either from a temporary recorder installation, or from observations of a local observer, and lag time could then be estimated. All aspects of this type of analysis should be carefully analyzed because it could lead to considerable error. It has been noted at regular gaging stations that time-to-peak varies considerably for individual storms. To establish the time-to-peak for unit hydrograph conditions, rainfall excess must occur within the unit duration, d, and be evenly distributed over the basin. These conditions seldom occur in nature.

For sites where time-to-peak has been established for an isolated storm, evenly distributed over the basin, and occurring within unit duration, the formula,

$$T_L = 2.4\, T_p^{0.8} + \frac{d}{2} \qquad (4)$$

may be used to estimate the adjusted lag time. It is best to use an average of several estimates of T_p. It can be seen that obtaining good estimates of T_p at ungaged sites may take considerable time, which in most applications will not be available.

Derivation of Synthetic Unit Hydrograph

A synthetic unit hydrograph for an ungaged site can be derived from the summation table (table 1) presented in this section. The variables necessary to make this derivation are, drainage area size, A, adjusted lag time, T_L, unit duration, d, and computation interval, Δt. (Computation interval, Δt, is selected to be equal to unit duration, d.) Table 1 is

T/T_L	Accumulated Percent 0	.01	.02	.03	.04	.05	.06	.07	.08	.09
0		.01	.03	.06	.10	.15	.21	.28	.36	.45
.1	.55	.66	.78	.90	1.03	1.17	1.32	1.48	1.65	1.83
.2	2.02	2.22	2.43	2.65	2.88	3.13	3.39	3.66	3.94	4.24
.3	4.55	4.88	5.22	5.57	5.94	6.33	6.73	7.15	7.59	8.04
.4	8.51	9.00	9.51	10.03	10.57	11.14	11.73	12.35	12.99	13.65
.5	14.33	15.04	15.77	16.52	17.29	18.08	18.90	19.74	20.60	21.48
.6	22.38	23.30	24.24	25.20	26.18	27.18	28.19	29.21	30.24	31.28
.7	32.33	33.39	34.46	35.53	36.60	37.68	38.76	39.84	40.92	42.01
.8	43.10	44.18	45.26	46.33	47.40	48.46	49.51	50.56	51.60	52.63
.9	53.64	54.63	55.61	56.57	57.52	58.45	59.37	60.27	61.15	62.02
1.0	62.87	63.71	64.53	65.34	66.13	66.90	67.66	68.40	69.13	69.84
1.1	70.54	71.22	71.88	72.52	73.15	73.76	74.36	74.94	75.50	76.05
1.2	76.58	77.10	77.61	78.11	78.60	79.08	79.55	80.01	80.46	80.90
1.3	81.33	81.75	82.16	82.56	82.95	83.34	83.72	84.09	84.45	84.81
1.4	85.16	85.50	85.83	86.16	86.48	86.79	87.10	87.40	87.70	87.99
1.5	88.27	88.55	88.82	89.09	89.35	89.61	89.86	90.11	90.35	90.59
1.6	90.82	91.05	91.27	91.49	91.70	91.91	92.11	92.31	92.50	92.69
1.7	92.87	93.05	93.23	93.41	93.58	93.75	93.92	94.08	94.24	94.40
1.8	94.55	94.70	94.85	94.99	95.13	95.27	95.40	95.53	95.66	95.79
1.9	95.91	96.03	96.15	96.27	96.38	96.49	96.60	96.71	96.81	96.91
2.0	97.01	97.11	97.20	97.29	97.38	97.47	97.55	97.63	97.71	97.79
2.1	97.87	97.95	98.02	98.09	98.16	98.23	98.30	98.36	98.42	98.48
2.2	98.54	98.60	98.66	98.71	98.76	98.81	98.86	98.91	98.96	99.01
2.3	99.06	99.10	99.14	99.18	99.22	99.26	99.30	99.34	99.38	99.41
2.4	99.44	99.47	99.50	99.53	99.56	99.59	99.62	99.65	99.67	99.69
2.5	99.71	99.73	99.75	99.77	99.79	99.81	99.83	99.85	99.86	99.87
2.6	99.88	99.89	99.90	99.91	99.92	99.93	99.94	99.95	99.96	99.97
2.7	99.98	99.99	99.99	99.99	99.99	99.99	99.99	99.99	99.99	99.99
2.8	100.00									

Table 1.--Summation table for synthetic unit hydrographs.

tabulated at .01 intervals of T/T_L, but to derive a smooth synthetic unit hydrograph, it is recommended that thousandths be used for values of T/T_L and that the table be interpolated.

The procedure for deriving a synthetic unit hydrograph is as follows:

1. Compute T/T_L for increments equal to Δt ($d = \Delta t$). The values of T/T_L should be listed up to and including the last value of T/T_L shown in the table.

2. Tabulate the corresponding percentages from the summation table. These are accumulated percentages for the desired unit hydrograph at intervals equal to Δt.

3. Take differences between succeeding values of the accumulated percentages. This gives the distribution, in percent, of the unit hydrograph for the selected unit duration and time interval. A plot of these values would yield a distribution graph.

4. To convert the distribution percentage to cubic feet per second (cfs), multiply each by the total cfs intervals, $\sum Q$, for one inch of runoff, computed by the formula,

$$\sum Q = \frac{645.3\ A}{\Delta t}$$

An example of the derivation of a unit hydrograph is given in the section, "Practical application".

BASE FLOW ESTIMATES

Base-flow estimates during floods consist in general of three parts: (1) an estimate of streamflow at the beginning of the storm period; (2) a base-flow recession curve; and (3) a transition between the initial estimate and the recession curve. The base-flow recession curve is probably the most important part of any estimate. Actual data should be used if available. Most applications, however, will probably be at ungaged sites where little or no information is available to determine base-flow recessions during floods. It will then become necessary to estimate the base-flow recession. To make this task as simple as possible, and to make all estimates consistent with the station data, average base-flow recessions for a range in drainage area sizes were determined from the station data. These curves are shown in figure 4. Average mergence points are shown for storms from 1 to 4 inches of runoff.

Each of the curves in figure 4 represent the average base-flow recession during and following storm-runoff periods for streams draining from 10 to 1300 square miles. The scale labeled "mergence point for runoff, in inches" denotes the total volume of surface runoff of the storm for which a base-flow recession is desired. The point at which the selected dashed curve intersects the base-flow recession curve is the point where surface runoff ceases. The segment of base-flow recession curve to the left of this point is the base-flow recession applicable to the storm in question.

The procedure for estimating base flow from the beginning to end of storm runoff is as follows:

1. A value of base flow at the beginning of storm runoff must be assumed. This value may be known at a gaged site, but for most applications it must be estimated. Generally a representative value of average low-flow conditions at the site can be used. If time and money permits, an actual low-flow measurement may be obtained. If no other means is available, initial base flow can be estimated as 0.5 cfs per square mile of drainage area.

2. The base-flow recession curve is determined from gaging station data if available; otherwise, the appropriate curve from figure 4 is selected. The last point, or mergence point, of the base-flow recession should coincide as closely as possible to the discharge indicated by the storm runoff. This point corresponds, in time, with the end of storm runoff.

3. The initial base flow assumed in (1) above is assumed to increase gradually during the beginning of storm runoff. At a point about halfway between the beginning of storm runoff and the peak of storm runoff, base flow is assumed to increase much more rapidly and at a point just beyond the peak, it starts to decrease at a rate indicated by the base-flow recession curve. The base-flow curve from beginning of storm runoff to a point just beyond the peak can be drawn as a smooth curve as described, merging with the base-flow recession curve determined previously. The sketch in figure 5 is a simplified example of a typical base-flow estimation from beginning to end of storm runoff.

The preceding example applies to single-peaked hydrographs produced by isolated storms. For multiple storms, runoff for each storm is considered separately and the resulting base-flow curves combined. For the purpose of base-flow application, a multiple storm occurs when there are two or more

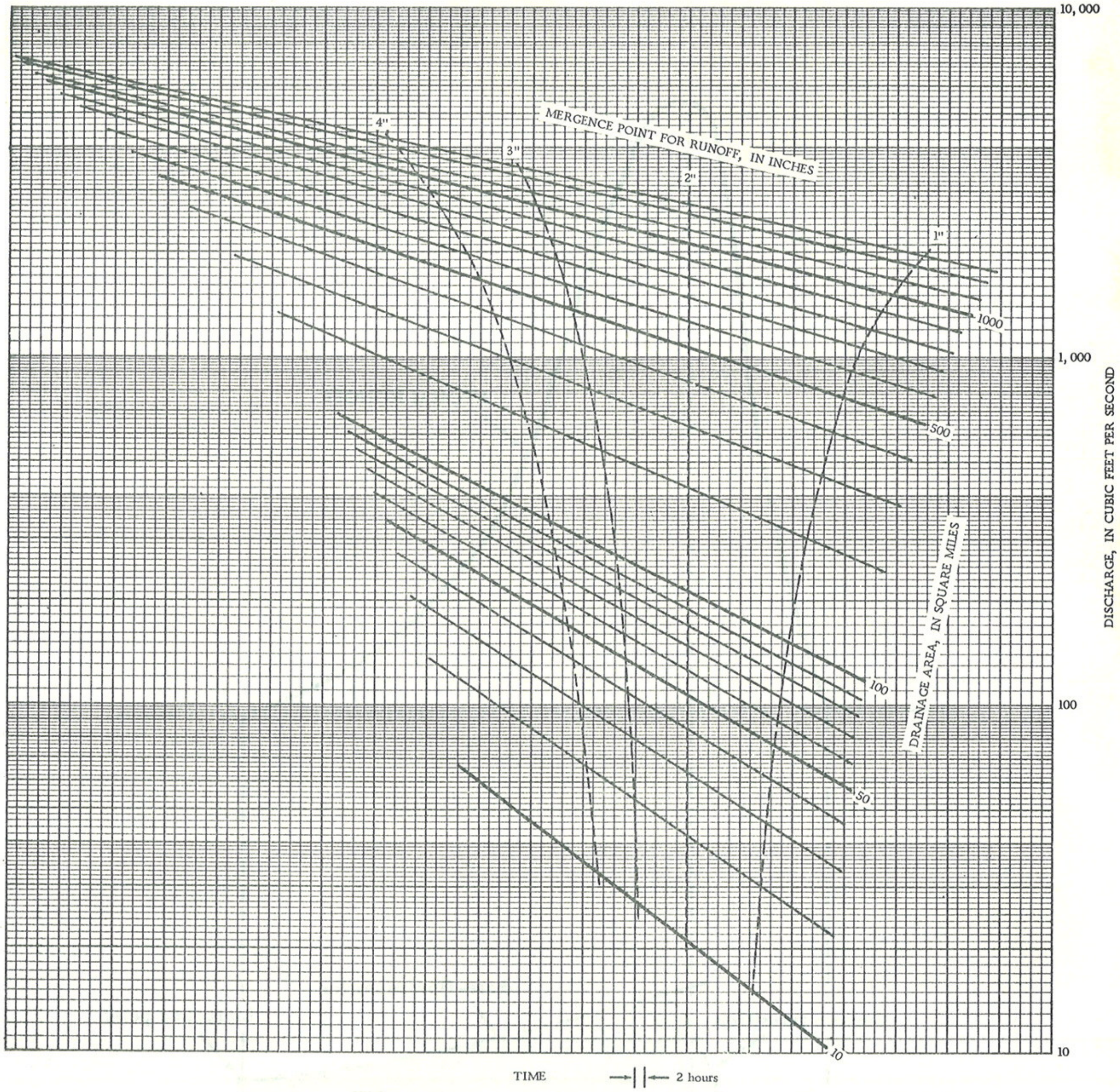


Figure 4.--Synthetic base-flow recession curves.

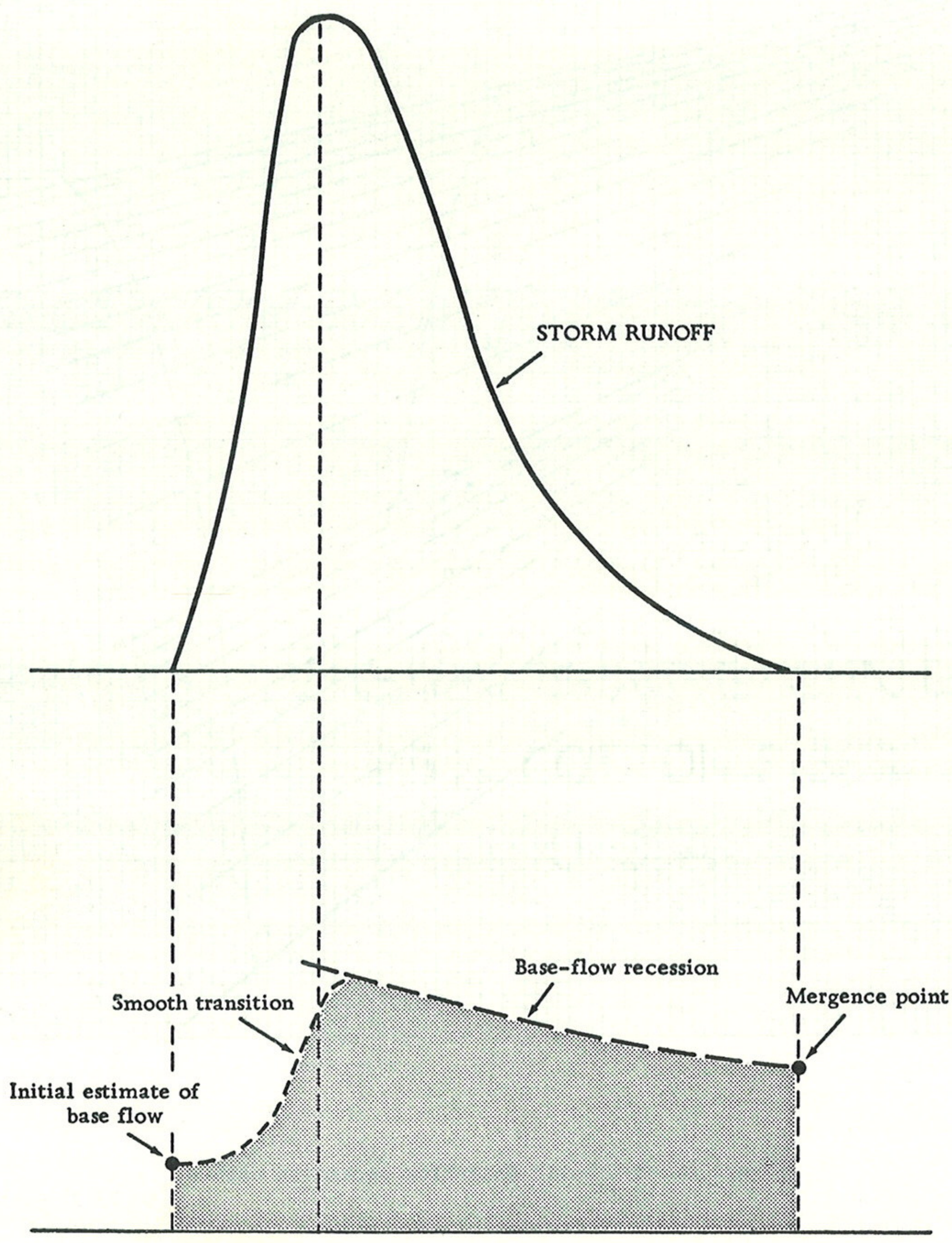


Figure 5.--Sketch showing application of base flow to isolated storm.

distinct runoff peaks, or when a distinct "hump" occurs in the storm-runoff hydrograph. Either case usually means that rainfall excess is broken into two or more portions. When this occurs, base flow must be applied on the basis of each portion of separate runoff. So many different combinations of multiple storms can occur that it is not practicable to show a solution for each. The following general rules can be used to combine most multiple storms into reasonable estimates of base flow:

1. When the runoff of the second storm equals or exceeds the runoff of the first storm, the base-flow recession of each storm is determined separately on the basis of the runoff for each storm. These recessions are then merged with a smooth transition as shown in figure 6.

2. When the runoff of the second storm is less than the first storm, the base-flow recession of the second storm will be either above or below the base-flow recession of the first storm. If it is above the first, the two can be merged as explained in (1) above and as shown in figure 6. If it is below, it is not logical to merge the two curves and it is recommended that the second base-flow recession curve be discarded and the first recession curve simply be extended downward at its normal rate.

3. When double peaks are the result of tributary timing, base flow should be applied as for an isolated storm.

Further examples of the application of base-flow recessions are given in the "Practical application" section. The user should not be concerned about extremely accurate definition of base flow, as long as fairly consistent methods are applied as described.

It is evident that in most cases, even large errors in base flow will not produce significant errors in total runoff, generally less than 5 percent.

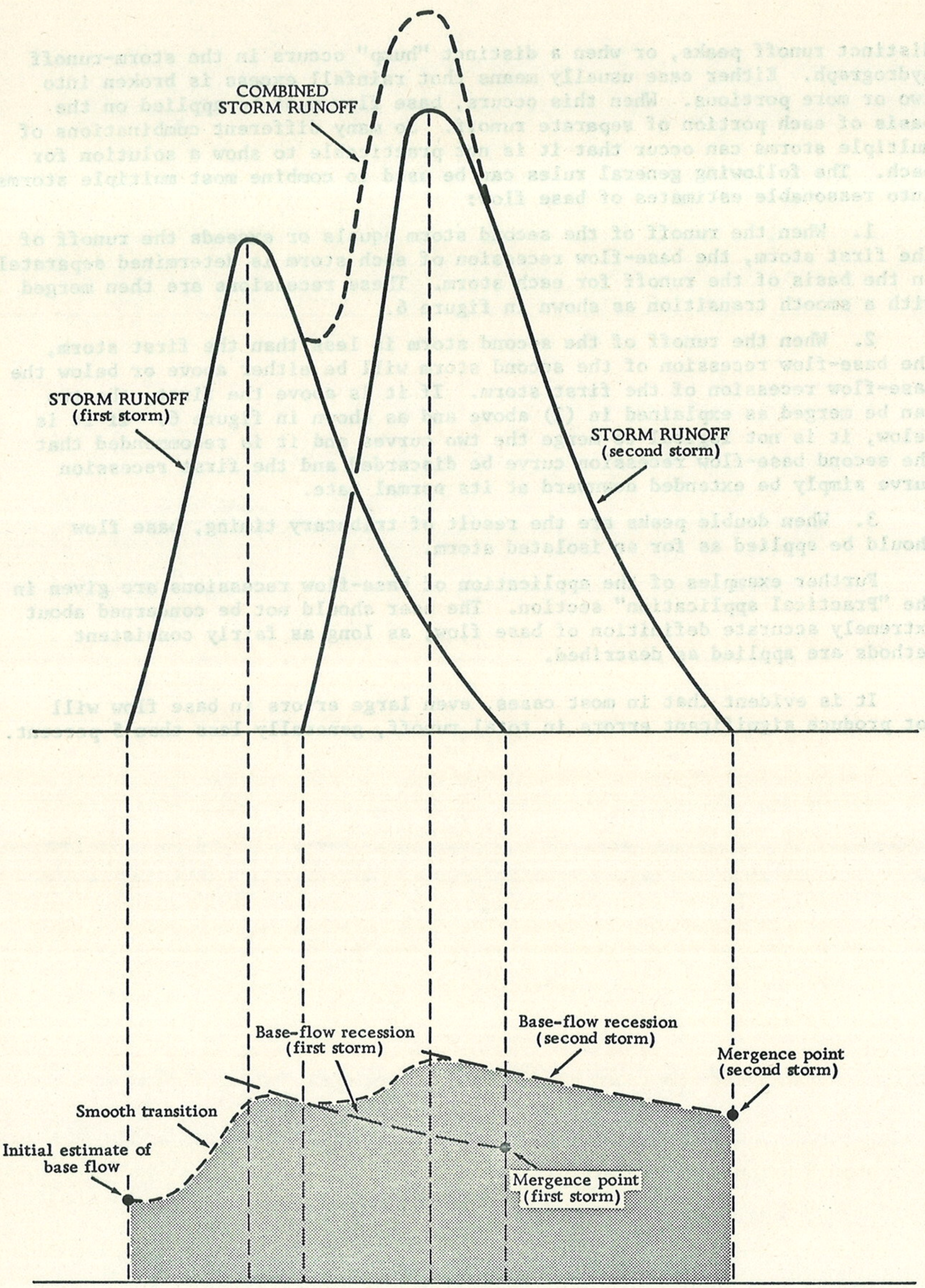


Figure 6.--Sketch showing application of base flow to multiple storms

PRACTICAL APPLICATION

The unit hydrograph may be used for several different situations to estimate flood hydrographs. For instance, it may be used for flood predictions, design of waterway structures and channels, estimation of missing streamflow records during flood periods, and extension of flood records on the basis of long term rainfall records. With proper precautions, it is a useful hydrologic tool and will undoubtedly find other uses as situations arise.

To assist the user in the application of the unit hydrograph, the following step-by-step summary lists the details which must be considered. In addition, detailed examples of specific situations are given.

Summary Procedure for Application of Unit Hydrographs

*1. From a good drainage map, determine the drainage area and mean length of the basin.

*2. Select a suitable unit duration, d.

*3. Estimate lag time.

4. Locate on the map all rainfall gages in or near the basin.

5. Determine Thiessen weight factors for each rain gage.

6. Compute average rainfall for time increments equal to d.

7. Compute rainfall excess from each increment of rainfall. (See Technical Report No. 2a)

*8. Derive the unit hydrograph in cfs.

9. If all rainfall excess is in one time increment equal to d, multiply each ordinate of the unit hydrograph by the rainfall excess to obtain a hydrograph of storm runoff. If the rainfall excess occurs in more than one time increment, the unit-hydrograph ordinates must be multiplied by each incremental rainfall excess, the resulting hydrographs lagged by the respective time differences, and summed. An example of such a computation is given in the following applications.

10. Plot the resulting hydrograph of storm runoff.

*11. Estimate base flow.

12. Sum storm runoff and base flow to obtain a hydrograph of total discharge for the storm period.

* Items already computed for regular gaging stations given in this report should be used in preference to synthetic methods.

Detailed Examples

Estimation of flood records at a gaging station.--This use of the unit hydrograph will be most helpful to those persons concerned with the collection and publication of gaging station records. To illustrate this example, the gaging station "Bogue Chitto near Bush, La." was selected. The storm beginning February 17, 1961, was used by assuming that the recorder stopped on February 16 and, consequently, all records of the flood were lost. The procedure of computing the storm by the unit-hydrograph method follows and the numbered sequence of steps is the same as given in the preceding "Summary procedure for application of Unit Hydrographs".

1-3. All of the factors listed in steps 1 through 3 are obtained from the gaging-station data listed on page 9.

(1) $A = 1{,}210$ square mile.
(1) $L_{ca} = 53.2$ miles (This item not used because lag time is known)
(2) $d = 6$ hours.
(3) $T_L = 65.8$ hours.

4-5. Thiessen weight factors were determined from rain gages in and around the basin.

6. The storm period was divided into three distinct occurrences: February 16 (12 p.m.) to February 18 (12 M); February 20 (6 p.m.) to February 22 (6 a.m.); and February 24 (6 a.m.) to Feb. 24 (6 p.m.). For purposes of computation, storm occurrences were started and ended on even 6 hour increments. To do this, rainfall records were adjusted slightly near the beginning and end of a storm; however, this has little effect on the accuracy of the final hydrograph.

The time distribution of rainfall at the recording gages was used as recorded. The distribution for stations reporting only daily totals was based on the nearest recording gages. Exact rules cannot be established for making a time distribution of rainfall because each problem involves a different set of conditions. Judgement should be used to obtain the best distribution from the available data.

After distributing the rainfall into six-hour increments for each rain gage, the Thiessen weight factors were applied and a total rainfall for the basin was determined for each six-hour increment.

It should be noted that slight deviations will occur when various individuals compute rainfall totals. This is only natural because of the judgement required in distributing the rainfall. Unless major deviations occur, final hydrographs will not differ widely.

7. Total rainfall is converted to runoff or rainfall excess according to procedures given in Technical Report No. 2a. The following table illustrates this procedure for this example.

Date	Ending time	Basin rainfall, inches	Accumulated rainfall, inches	*Accumulated runoff, inches	Runoff differences, inches
Feb. 17	6 a.m.	0.61	0.61	0.27	0.27
	12 m	0	0.61	0.27	0
	6 p.m.	2.00	2.61	1.49	1.22
	12 p.m.	1.65	4.26	2.66	1.17
Feb. 18	6 a.m.	0	4.26	2.66	0
	12 m	0.29	4.55	2.87	0.21
Feb. 20	12 p.m.	0.83	0.83	0.38	0.38
Feb. 21	6 a.m.	0	0.83	0.38	0
	12 m	2.50	3.33	1.99	1.61
	6 p.m.	0.30	3.63	2.20	0.21
	12 p.m.	1.02	4.65	2.95	0.75
Feb. 22	6 a.m.	1.00	5.65	3.71	0.76
Feb. 24	12 m	1.17	1.17	0.58	0.58
	6 p.m.	.94	2.13	1.17	0.59

* From technical report 2a, accumulated runoff = aP^x, where a and x are coefficients dependent on the week, and P is accumulated rainfall. For week 8 (Feb. 17-24), a = 0.480 and x = 1.181.

8. The unit hydrograph data for this station are given on page 9. The time increment between successive values of discharge is 6 hours.

9. Each ordinate of the unit hydrograph is multiplied by each value of runoff, in inches, as computed in item 7 above. The resulting values are listed in tabular form, each column beginning at the corresponding time of rainfall. After each increment of runoff has been accounted for, horizontal totals will yield the storm runoff hydrograph. Table 2 illustrates this procedure for this example.

10. Storm runoff is plotted as shown in figure 7.

11. Base flow is divided into 3 distinct parts corresponding to the 3 storm periods. Initial base flow (1,120 cfs) is that which was flowing at the time the record is assumed to have stopped on February 16. Base-flow recessions are based on the station data given on page 9.

The initial flow (1,120 cfs) is merged into the base-flow recession curve of the first storm by a smooth transition as described on page 20, step 3. The base-flow recessions of the first and second storms are merged together with a smooth transition. The base-flow recession of the third storm, being lower than that of the second storm, is discarded and the base-flow recession of the second storm is extended downward at its normal rate. See figure 7 for the complete base flow estimation.

After base flow is drawn in as shown on figure 7, tabulate values in table 2.

		Runoff, multiply each by unit hydrograph ordinates											Totals		
Date	Time	0.27	1.22	1.17	0.21	0.38	1.61	0.21	0.75	0.76	0.58	0.59	Storm Runoff, cfs	Base Flow, cfs	Total Runoff, cfs
Feb 17	6a	14											14	1120	1130
	12M	28											28	1120	1150
	6p	99	63										162	1120	1280
	12p	421	127	61									609	1200	1810
18	6a	1040	445	122									1610	1600	3210
	12M	1650	1900	427	11								3990	2100	6090
	6p	2350	4680	1830	22								8880	2900	11800
	12p	3290	7470	4490	77								15300	3900	19200
19	6a	4100	10600	7160	328								22200	5300	27500
	12M	4270	14900	10200	806								30200	6200	36400
	6p	3940	18500	14300	1280								38000	6800	44800
	12p	3460	19300	17800	1830								42400	7300	49700
20	6a	2860	17800	18500	2560								41700	7700	49400
	12M	2140	15600	17100	3190								38000	7600	45600
	6p	1580	12900	15000	3320								32800	7200	40000
	12p	1160	9670	12400	3070	19							26300	6800	33100
21	6a	826	7150	9280	2690	40							20000	6400	26400
	12M	597	5250	6860	2230	139	84						15200	6000	21200
	6p	440	3730	5030	1670	593	167	11					11600	5600	17200
	12p	316	2700	3580	1230	1460	588	22	39				9940	5300	15200
22	6a	211	1990	2590	903	2320	2510	77	78	40			10700	5200	15900
	12M	141	1430	1910	643	3310	6180	328	274	79			14300	5200	19500
	6p	88	954	1370	464	4640	9850	806	1170	277			19600	5200	24800
	12p	53	636	915	342	5780	14000	1280	2880	1180			27100	5400	32500
23	6a	35	396	610	246	6000	19600	1830	4590	2920			36200	5800	42000
	12M	18	238	380	164	5550	24500	2560	6540	4650			44600	6400	51000
	6p		159	228	109	4860	25400	3190	9150	6630			49700	7200	56900
	12p		81	152	68	4030	23500	3320	11400	9270			51800	7700	59500
24	6a			77	41	3010	20600	3070	11800	11600			50200	8100	58300
	12M				27	2230	17100	2690	11000	12000	30		45100	8400	53500
	6p				14	1630	12800	2230	9600	11100	60	31	37500	8500	46000
	12p					1160	9430	1670	7950	9730	212	61	30200	8000	38200
25	6a					840	6920	1230	5950	8060	905	215	24100	7600	31700
	12M					619	4930	903	4400	6030	2230	920	20000	7200	27200
	6p					445	3560	643	3220	4450	3550	2270	18100	6800	24900
	12p					297	2620	464	2300	3270	5060	3610	17600	6400	24000
26	6a					198	1880	342	1660	2320	7080	5140	18600	6000	24600
	12M					124	1260	246	1220	1680	8820	7200	20600	5600	26200
	6p					74	839	164	878	1240	9160	8970	21300	5300	26600
	12p					49	523	109	587	890	8470	9320	19900	5000	24900
27	6a					25	314	68	391	594	7420	8610	17400	4700	22100
	12M						209	41	244	396	6150	7550	14600	4400	19000
	6p						106	27	146	247	4600	6250	11400	4200	15600
	12p							14	98	148	3400	4680	8340	4000	12300
28	6a								50	99	2490	3460	6100	3700	9800
	12M									50	1770	2540	4360	3500	7860
	6p										1280	1810	3090	3300	6390
	12p										945	1300	2240	3100	5340
Mar 1	6a										679	962	1640	3000	4640
	12M										454	690	1140	2800	3940
	6p										302	461	763	2600	3360
	12p										188	307	495	2500	3000
2	6a										113	192	305	2400	2700
	12M										75	115	190	2200	2390
	6p										38	77	115	2100	2220
	12p											39	39	2000	2040

Table 2.--Computation of storm runoff, storm of Feb. 17 - Mar. 2, 1961, Bogue Chitto near Bush, La.

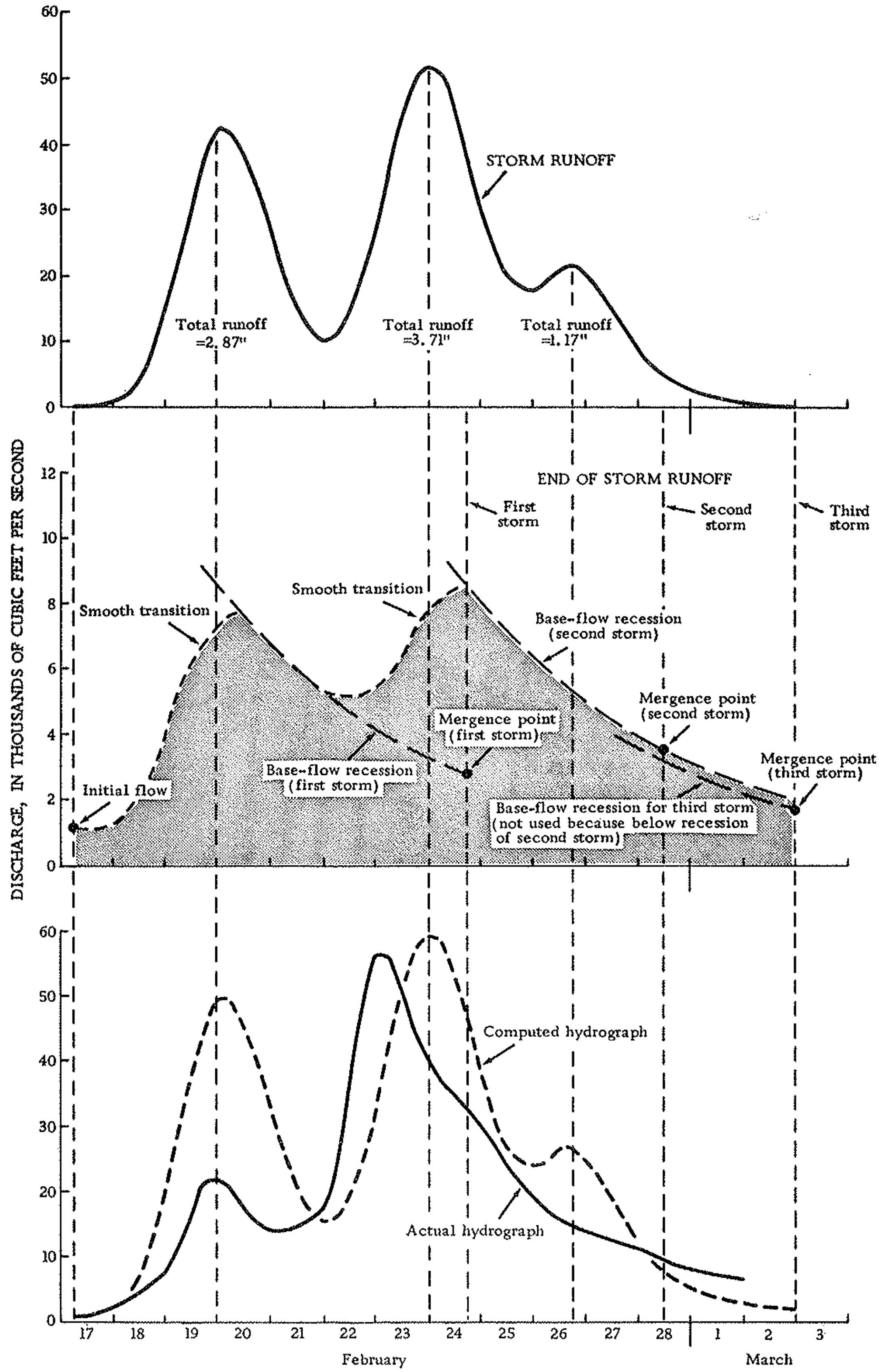


Figure 7.--Estimated storm hydrograph and base flow, Bogue Chitto near Bush, La., period Feb. 17 - Mar. 2, 1961

12. Storm runoff and base flow are then summed in the table, and these values are then plotted to obtain the computed hydrograph (fig. 7). Also shown on this figure is the actual hydrograph for comparison. Actual and computed hydrographs compare favorably for the second storm, but differ appreciably for the first storm. This difference can be ascribed to several causes as explained previously. Unequal distribution of the rainfall over the basin, insufficient number of rain gages, inaccuracies in converting rainfall to rainfall excess, and unit-hydrograph assumptions not completely met are the major causes of error. Base-flow estimations, although poor, generally do not cause large errors in the final flood hydrograph.

<u>Computation of a hypothetical storm</u>.--Designers of structures such as dams, highway bridges and levees are often interested in the characteristics of hydrographs resulting from hypothetical storms. Here again the unit hydrograph is a useful tool.

This example will be used to demonstrate the computation of the hydrograph resulting from a hypothetical 100-year, 24-hour rainfall, evenly distributed over the Comite River basin. The site chosen for this example is State Highway 946, which is the location of the gaging station "Comite River near Comite, La." (see p. 12). The following computations are based on station data and follow the same sequence as shown on page 25.

1-3. The factors listed in items 1 through 3 are obtained from page 12 and are as follows:

A = 284 square miles

L_{ca} = 21.3 miles (this item not used in this example because lag time is known).

d = 4 hours

T_L = 37.3 hours

4-7. In this example rainfall is assumed to be uniformly distributed over the basin and also uniformly distributed during 24 hours, therefore, Thiessen weight factors are not necessary. The hypothetical 100-year storm is taken from U.S. Weather Bureau Technical Report No. 40 and was adjusted for size of basin. The 100-year, 24-hour point rainfall is 11.5 inches (chart 49 of that report). The adjustment factor for 284 square miles is 0.92 (see fig. 15 of that report). The average rainfall over the basin equals 0.92 times 11.5, or 10.6 inches. Chart 54 of that report shows that the most likely time of occurrence is either April or September to October. Based on rainfall-runoff relations, the April occurrence will yield the greater runoff (week 14 used). Rainfall and runoff distributions are given in the following table:

Time	Rainfall, inches	Accumulated rainfall, inches	*Accumulated runoff, inches	*Runoff, inches
1st-4 hours	1.77	1.77	0.88	0.88
2nd-4 hours	1.76	3.53	2.02	1.14
3rd-4 hours	1.77	5.30	3.31	1.29
4th-4 hours	1.77	7.07	4.69	1.38
5th-4 hours	1.76	8.83	6.12	1.43
6th-4 hours	1.77	10.60	7.68	1.56

* See Technical Report No. 2a for method of computing runoff.

8. The unit hydrograph is used as given on page 12 with the time increment, Δt, equal to 4 hours.

9. Runoff for each time increment is multiplied by each value of the unit hydrograph as shown in table 3. Horizontal totals yield the storm runoff hydrograph.

10. The hydrograph is plotted as shown in figure 8.

11. Base flow is estimated as shown in figure 8.
    Note that the mergence point for 7.68 inches of runoff is not shown on page 12; therefore, it was estimated to be 1,030 cfs on the basis of the other mergence points. Slight errors here will not be critical. The initial base flow is determined from station records to be about 100 cfs (average low flow in April). Base flow is picked from the final curve (fig. 8) and tabulated in the table.

12. Storm runoff and base flow are then summed to obtain the total hydrograph. A plot of the total runoff is shown in figure 8.

Derivation of a synthetic unit hydrograph.--A synthetic unit hydrograph can be derived for any site in the study area by using the procedure given on page 19. The following example is given as an illustration for one site.

To compute a synthetic unit hydrograph, certain basin characteristics must be known. Drainage area size, in square miles, and adjusted lag time, in hours, are the two factors necessary. Adjusted lag time can be computed from mean length of the basin as explained on page 14. For this example, suppose it is desired to compute a synthetic unit hydrograph for Bogue Lusa Creek at State Highway 10. (Assume for the moment that this is an ungaged site). The drainage area is computed to be 12.1 square miles and the mean length 2.7 miles. The adjusted lag time, T_L is computed from formula (2) on page 15. Formula (2) is used because Bogue Lusa Creek is in sub-area 2.

$$T_L = 2.8\, L_{ca}^{0.8} + \frac{d}{2} \qquad (2)$$

$$T_L = 2.8\,(2.7)^{0.8} + \frac{d}{2}$$

$$T_L = 6.2 + \frac{d}{2} \quad \text{(partial computation of } T_L\text{)}$$

		Runoff, multiply each by unit hydrograph ordinates						Totals, in cfs		
Date	Time	0.88	1.14	1.29	1.38	1.43	1.56	Storm Runoff	Base Flow	Total Runoff
April of any year most likely	4-hour intervals	141						141	100	241
		924	182					1110	120	1230
		1970	1200	206				3380	170	3550
		2660	2550	1350	221			6780	200	6980
		3000	3440	2890	1450	229		11000	300	11300
		3200	3890	3900	3090	1500	250	15800	430	16200
		3330	4150	4400	4170	3200	1640	20900	630	21500
		3390	4310	4700	4710	4320	3490	24900	920	25800
		3360	4390	4880	5020	4880	4710	27200	1300	28500
		3220	4350	4970	5220	5210	5320	28300	1700	30000
		2980	4170	4930	5310	5410	5680	28500	1900	30400
		2660	3860	4720	5270	5510	5900	27900	2050	30000
		2300	3440	4370	5050	5460	6010	26600	2150	28800
		1970	2980	3900	4680	5230	5960	24700	2200	26900
		1590	2550	3370	4170	4850	5710	22200	2200	24400
		1250	2060	2890	3600	4320	5290	19400	2200	21600
		906	1620	2330	3090	3730	4710	16400	2100	18500
		607	1170	1830	2500	3200	4070	13400	2000	15400
		387	787	1330	1960	2590	3490	10500	1900	12400
		220	502	890	1420	2030	2820	7880	1800	9680
		123	285	568	952	1470	2220	5620	1700	7320
		62	160	322	607	987	1610	3750	1600	5350
		44	80	181	345	629	1080	2360	1500	3860
		18	57	90	193	358	686	1400	1430	2830
			23	64	97	200	390	774	1340	2110
				26	69	100	218	413	1280	1690
					28	72	109	209	1200	1410
						29	78	107	1150	1260
							31	31	1080	1110
								0	1030	1030

Table 3.--Computation of runoff for hypothetical 100-year, 24-hour storm, Comite River near Comite, La.

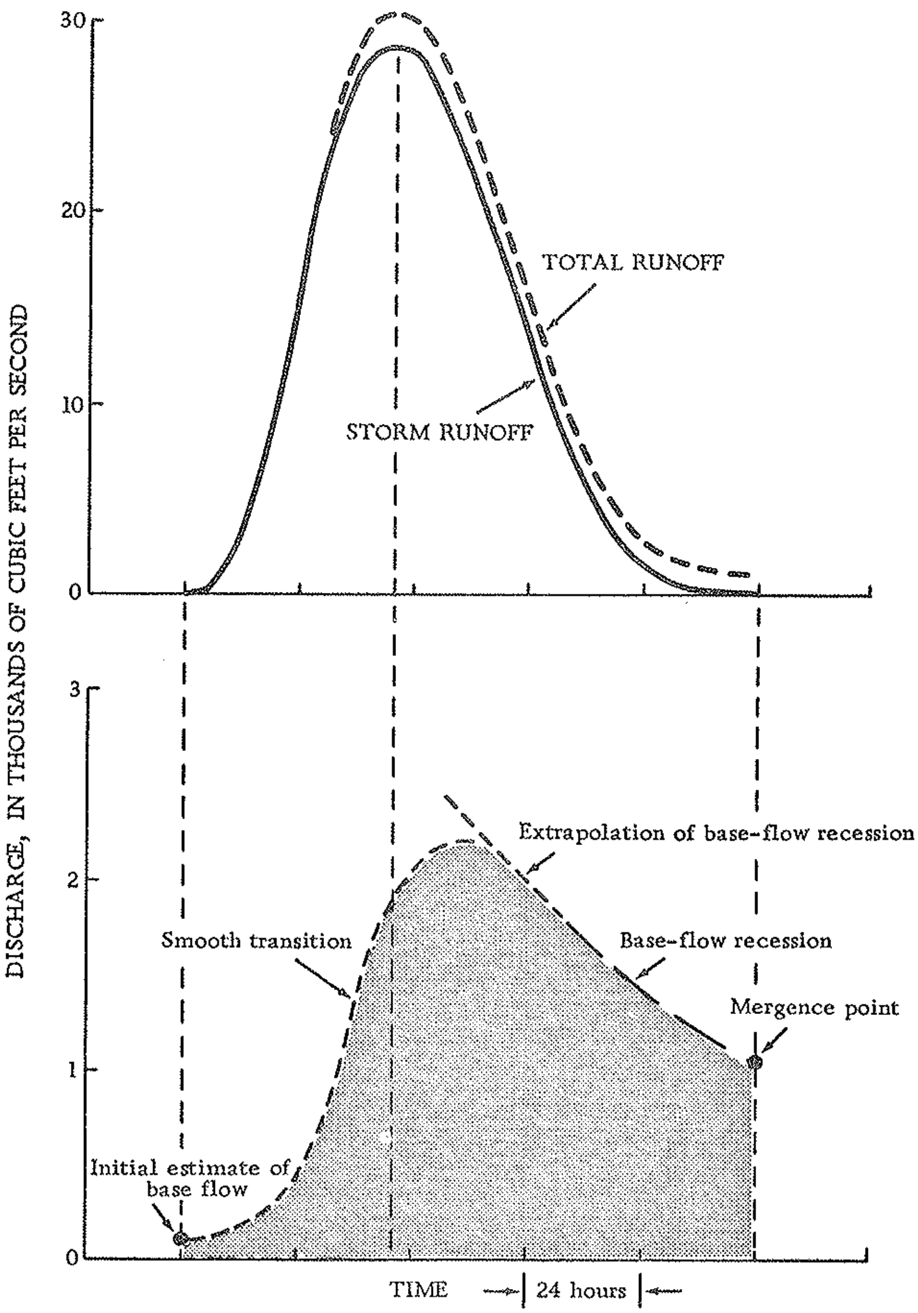


Figure 8.--Hypothetical flood hydrograph derived from 100-year, 24-hour storm, Comite River near Comite, La.

It can be seen at this point that T_L will probably be less than 8, therefore, from the table on page 14, d is selected to be 1 hour. T_L can now be computed as follows:

$$T_L = 6.2 + \frac{d}{2}$$

$$T_L = 6.2 + \frac{1}{2}$$

$$T_L = 6.7 \text{ hours}$$

From this point, the computations follow the numbered sequence beginning on page 19.

1. The computation interval, Δt, is selected to be equal to d, or 1 hour. Values of T/T_L are then computed and listed in tabular form as shown in column 2 of table 4. These are computed up to a value of T/T_L equal to 2.8.

Table 4.--Computation of synthetic unit hydrograph, Bogue Lusa Creek near Franklinton, La.

(1) Time, T (hours)	(2) $^T/_{T_L}$	(3) Accumulated percent	(4) Difference, percent	(5) Unit hydrograph, cfs
0	-	-	-	-
1	0.149	1.16	1.16	91
2	.299	4.52	3.36	262
3	.448	11.03	6.51	508
4	.597	22.11	11.08	865
5	.747	37.36	15.25	1190
6	.896	53.24	15.88	1240
7	1.045	66.57	13.33	1040
8	1.194	76.26	9.69	757
9	1.344	83.11	6.85	535
10	1.493	88.07	4.96	387
11	1.642	91.74	3.67	287
12	1.792	94.43	2.69	210
13	1.941	96.39	1.96	153
14	2.090	97.79	1.40	109
15	2.240	98.76	.97	76
16	2.389	99.41	.65	51
17	2.538	99.79	.38	30
18	2.687	99.97	.18	14
19	2.837	100.0	.03	2

2. Accumulated percentages are then obtained from table 1 corresponding to the computed values of T/T_L and tabulated in table 4 as shown in column 3. Interpolation must be used to produce a smooth hydrograph.

3. The unit hydrograph, in percent, is then obtained by taking differences of succeeding values obtained in step 2 above. These differences are shown in column 4 of table 4.

4. The percentages are then converted to discharge, in cfs, by multiplying each by the value of $\sum Q$ for 1 inch of runoff. $\sum Q$ is computed as follows:

$$\sum Q = \frac{645.3A}{\Delta t}$$

$$\sum Q = \frac{645.3\ (12.1)}{1}$$

$$\sum Q = 7{,}808 \text{ cfs intervals (in this case cfs hours)}$$

Data for the synthetic unit hydrograph, in cfs, are shown in column 5 of table 4. A comparison with the actual unit hydrograph is shown in figure 9.

ACCURACY AND LIMITATIONS

The accuracy of the methods and procedures given in this report are difficult to evaluate because of the many variables involved. Individual site data are the best data to use when deriving hydrographs for one of the sites listed in this report. The synthetic derivations cannot improve the site data, and they certainly will not reproduce the abnormal type of hydrograph that occasionally occurs at some sites. Sufficient rainfall data and the reduction of those data to rainfall excess play a big part in the final accuracy of any particular problem. In short, there is no way to predict the accuracy of results nor to give accuracy limits.

All the unit hydrograph data presented on pages 9-12 were tested by actually reproducing large storms. These were tests of the unit hydrograph only; rainfall excess was adjusted to equal the actual measured runoff. Good results were obtained in all cases.

The regionalized data were tested against the actual data by comparing synthetically derived unit hydrographs to the actual unit hydrograph data given on pages 9-12. To compute the synthetic unit hydrograph, lag time was computed by using the appropriate formula based on mean length as shown on page 15. A unit hydrograph was then computed on the basis of this lag time and the summation table (page 18). A graphical comparison is shown in figures 9a-9s. Generally good comparisons are evident from these plots. It should be noted that this is a combination test of the synthetic unit hydrograph and the computation of lag time. It illustrates the results that would be obtained if nothing other than basin parameters were known. Of course the accuracy of the synthetic unit hydrograph depends considerably on the accuracy of the computed lag time. The user should be familiar with the type of streams in the study area and be alert for conditions which may deviate from those for which the formulas were derived.

Accuracy of base-flow recessions, estimated from either actual data or regionalized data, is questionable even in its broadest interpretation; however, large errors in estimating base flow usually will not greatly affect the overall accuracy. The main objective is to provide a consistent method for completing the final hydrograph after storm runoff has been computed. Although the user may disagree with the method of applying base flow, he should bear in mind that the methods presented in this report were the same as those used to derive the unit hydrographs. The use of a radically different method of applying base flow might lead to larger errors than are already inherent in the methods presented.

In summary, the data and procedures of this report can be used for practical application of the unit hydrograph to streams in the study area. The following limitations should be observed:

(1) Before using the unit-hydrograph method it should be ascertained that the general assumptions (see page 6) are met reasonably well. Naturally, fulfillment in every detail of these assumptions cannot be required or the methods could never be used. As a general rule, it can be assumed that the greater the deviation from the basic assumptions, the greater the error in the final hydrograph. In some instances, adjustments for these deviations can be made, and this should be done whenever it is deemed necessary.

(2) The method has not been tested for sites of less than about 10 square miles drainage area; therefore if it is used for small areas, there is no assurance that large errors will not occur.

(3) The regionalized data should be used only within the study area although there is some evidence that a synthetic unit hydrograph can be computed for any site for which lag time is known. This was observed by successfully reproducing unit hydrographs for Illinois streams. It was further observed that the synthetic unit hydrograph methods of this report give almost identical results to those obtained by a method called the "model hydrograph" derived by Mitchell, (report in publication).

(4) The formulas for computing lag time should definitely not be used outside the study area. These formulas were derived strictly for streams within the study area and streams outside the area will undoubtedly have different travel-time characteristics. In fact, there may be some streams in the study area, which have not been previously gaged, that show altogether different characteristics from those defined by the formulas.

Results obtained by using methods of this report will generally be acceptable for most engineering work involving computations of storm hydrographs. It should be emphasized, however, that the user cannot expect exact reproductions of known hydrographs nor should he expect predicted results to be exact. He should also expect to find streams in the study area which have different characteristics from streams studied to date. Adjustments should be made whenever there is sufficient basis for doing so.

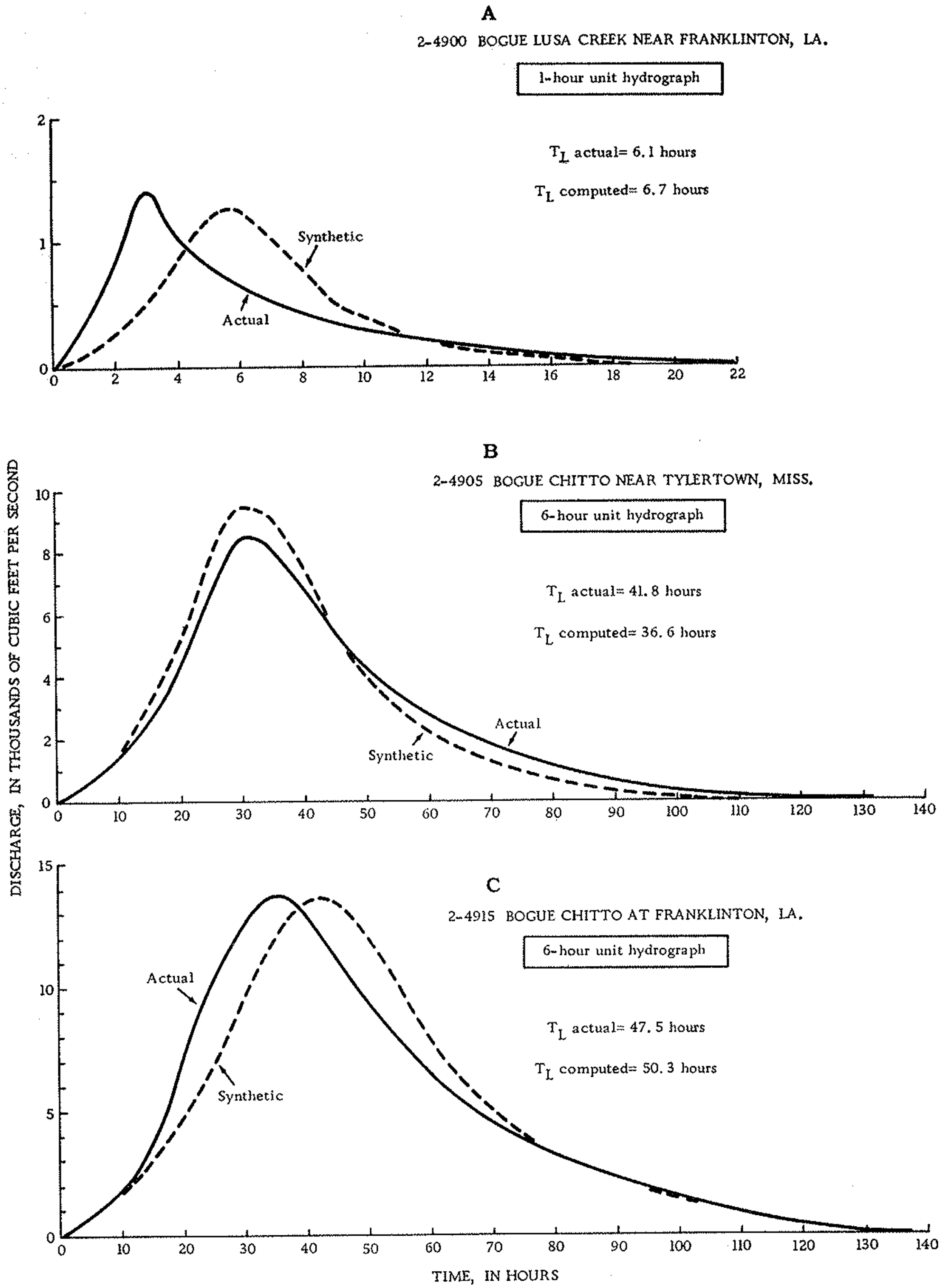


Figure 9.--Comparisons of actual unit hydrographs to synthetic unit hydrographs.

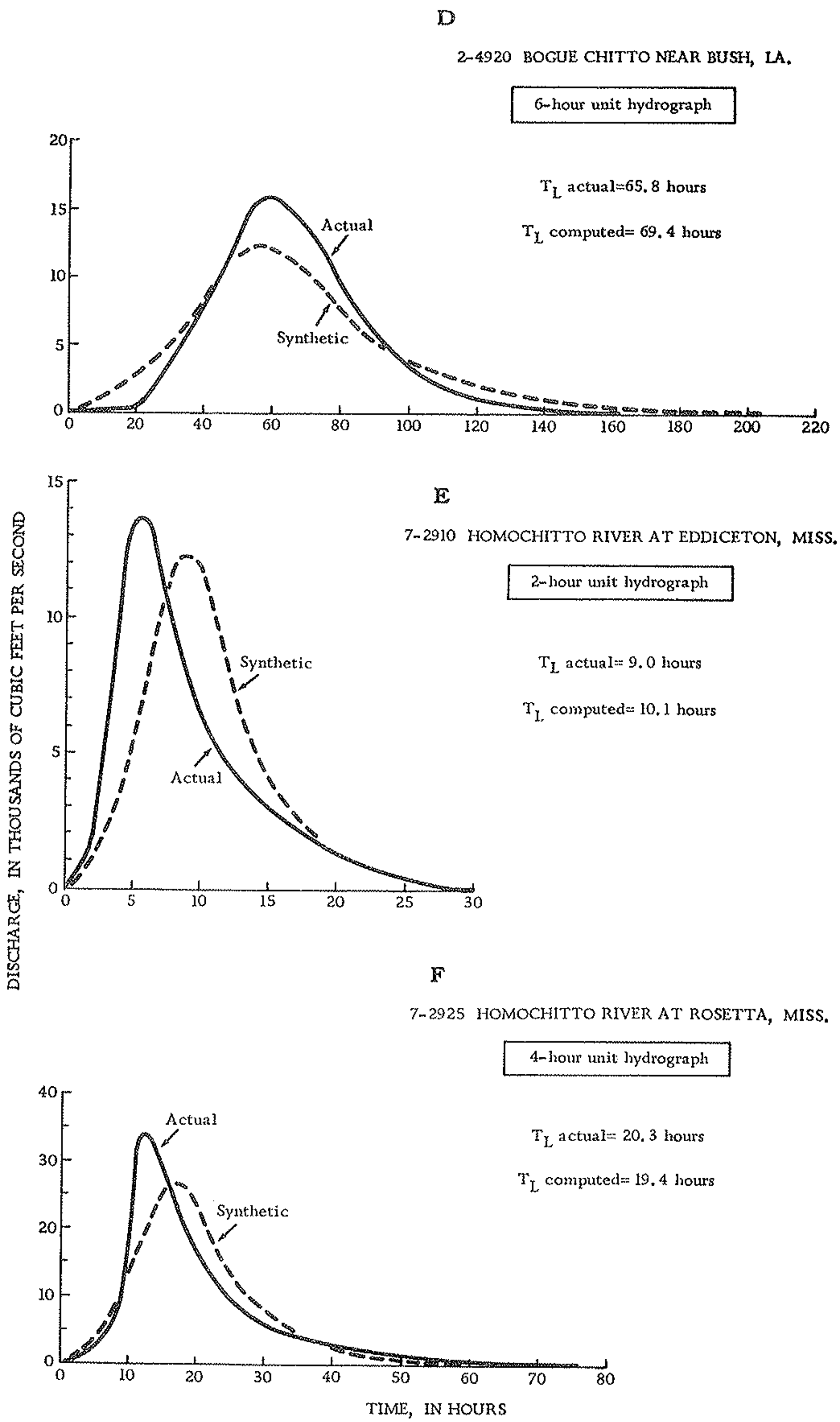


Figure 9 (continued).--Comparisons of actual unit hydrographs to synthetic unit hydrographs.

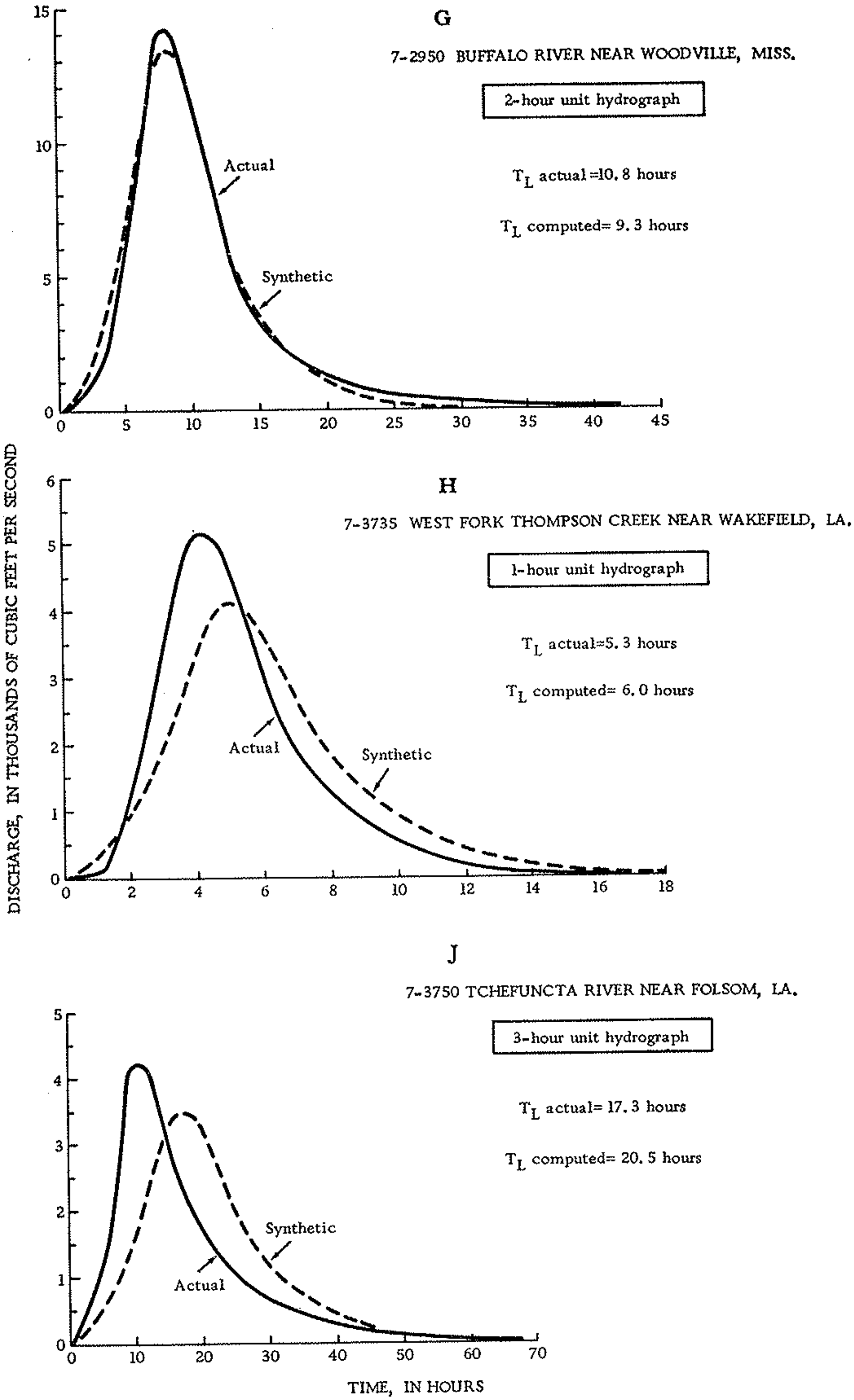


Figure 9 (continued).--Comparisons of actual unit hydrographs to synthetic unit hydrographs.

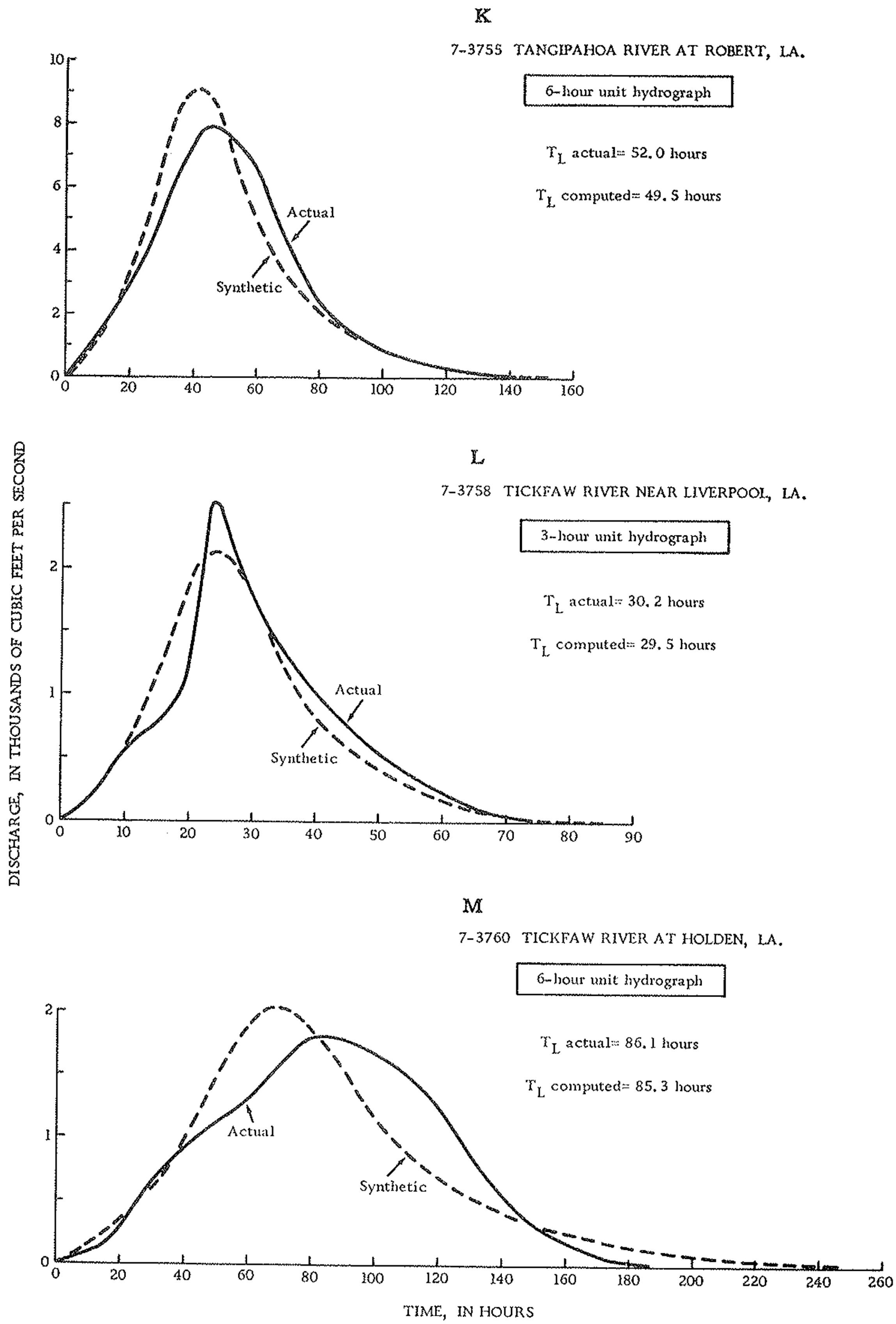


Figure 9 (continued).--Comparisons of actual unit hydrographs to synthetic unit hydrographs.

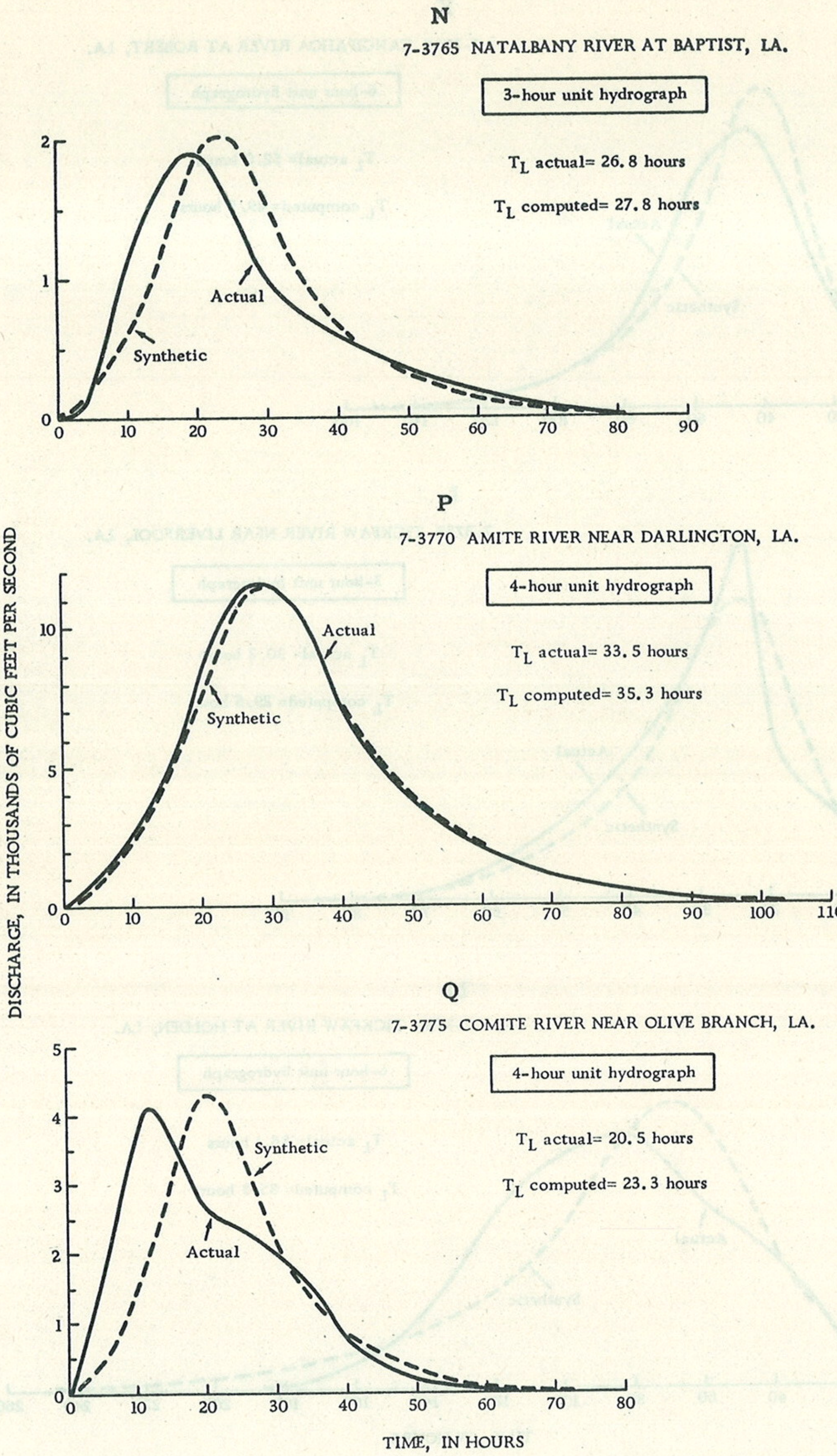


Figure 9 (continued).--Comparisons of actual unit hydrographs to synthetic unit hydrographs.

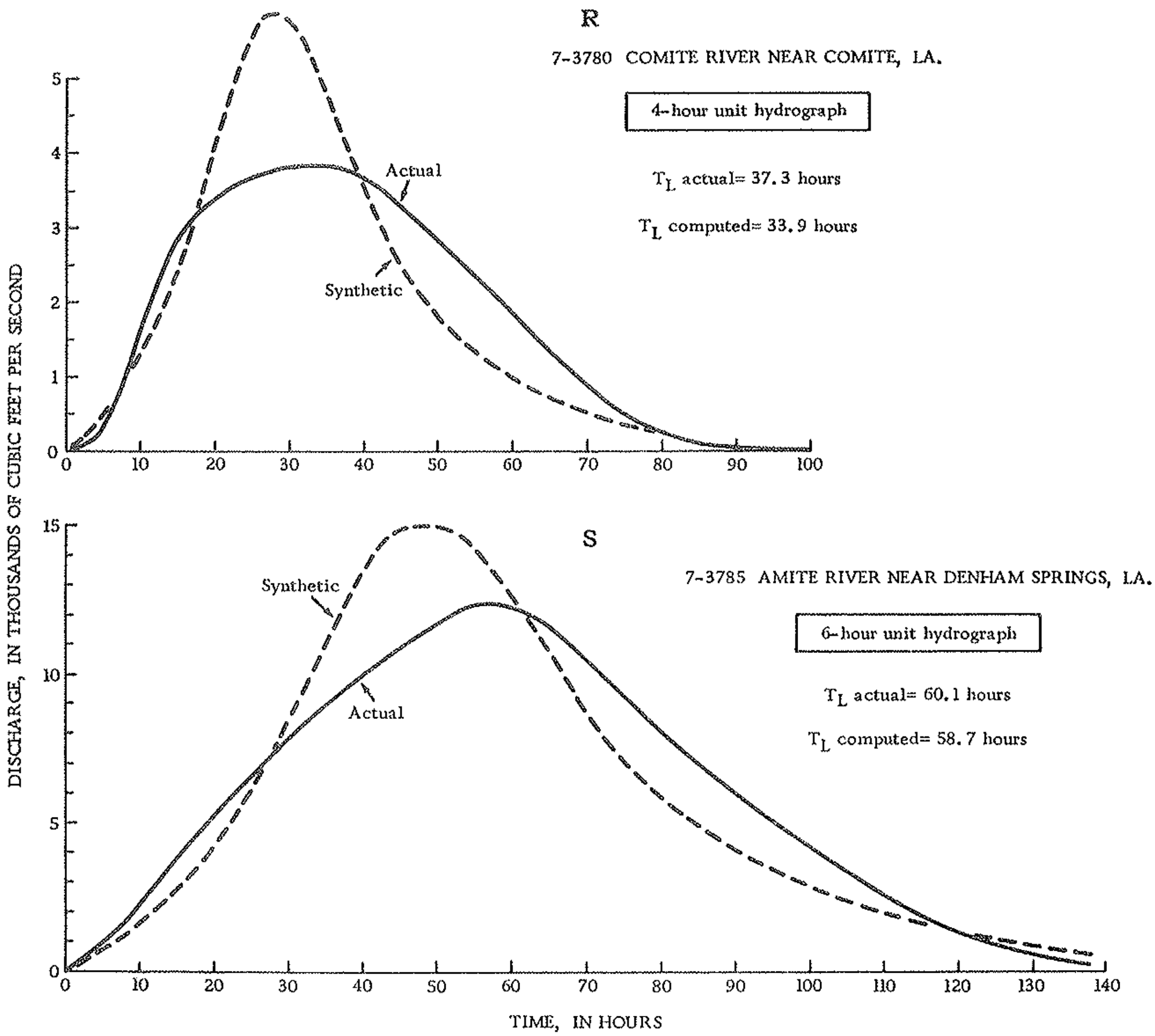


Figure 9 (continued).--Comparisons of actual unit hydrographs to synthetic unit hydrographs.

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APPENDIX

Definition of Terms

Area: See "drainage area".

Base flow, in cfs: Generally referred to as the amount of flow which enters a stream through the bed and banks, as opposed to the flow which enters as surface runoff. Specifically base flow can be spring flow, seepage, subsurface flow, or combinations of these; however, the purposes of this report do not require a separation of base flow into its components.

Base flow recession: The rate at which base flow recedes following storm runoff. Base-flow recession data for a particular site are given as a series of discharges at selected time intervals. Although it is known that such recessions vary with the season of the year, only average conditions are considered necessary for application of this report.

Computation interval (Δt), in hours: The interval of time between successive computations of a particular problem. For a unit hydrograph, Δt is equal to unit duration, d.

Discharge (Q), in cubic feet per second (cfs): The rate of flow at a particular instant of time.

Distribution graph: A flood hydrograph in which the ordinates have been expressed as percentages of their sum.

Drainage area (A), in square miles: The total surface area contributing to the surface drainage of a basin.

Duration, unit (d), in hours: See "Unit duration".

Flood routing: A process of predicting, or estimating, the flood hydrograph at some point on a stream from data for an upstream location. The process takes into account inflow and storage.

Hydrograph: A plot of discharge (ordinate) versus time (abcissa).

Isolated storm: A storm occurring at a time when streamflow is all base flow and from which runoff recedes before another storm occurs. See also "multiple storm".

Lag time, adjusted, (^{T}L), in hours: Lag time plus d/2. In effect, adjusted lag time is the time measured from beginning of rainfall excess to the center of mass of runoff for the unit hydrograph.

Lag time, in hours: The time measured from the center of mass of rainfall excess to the center of mass of the resulting runoff. Lag time was computed from the final unit hydrograph derived for each station. Center of mass of rainfall excess for a unit hydrograph located with respect to time, is one-half of its duration, ($^{d}/_{2}$) from its beginning. Center of mass of runoff is determined by multiplying each ordinate of the unit hydrograph by its interval (in hours) from the beginning of runoff and dividing the sum of these products by the sum of the ordinates.

Length, basin (L), in miles: The distance from a designated point on a stream to the surface-drainage divide. Basin length is measured along the main stem and follows the general trend of the floodplain rather than the meandering low-water channel.

Length, basin mean (L_{ca}), in miles: The average distance which flood water must travel within a basin to reach the outlet. The distance is measured along the general path of the floodplain and is not representative of low-water channel distances.

Mean length, basin (L_{ca}) in miles: See "Length, basin mean".

Multiple storm: A storm involving separate periods of rainfall so closely spaced in time that runoff from one combines with runoff from another. A multiple storm generally produces more than one discharge peak during the combined flood period. See also "Isolated storm".

Rainfall excess: The volume of rainfall available for direct runoff; the residual of rainfall, after all losses such as interception, infiltration, evapotranspiration and surface storage have been satisfied. See also "Runoff".

Routing, flood: See "Flood routing".

Runoff (R), in inches, or (ΣQ), in cfs - intervals: The total rainfall excess resulting from an individual storm. Although runoff can be expressed in other volumetric dimensions, inches and cfs-intervals are the two used for this report. Runoff in inches is the depth of water which would result if the total volume were spread evenly over the whole drainage basin. Cfs-intervals is the volume expressed in the same time dimension as used for the computation interval of a particular problem. See the section "Derivation of Synthetic Unit Hydrograph" for computation of runoff volume in cfs-intervals.

Summation curve: A flood hydrograph with discharge accumulated at equal time intervals. Discharge for such a curve may be expressed in any convenient units, but generally is expressed in percent or cfs.

Summation table: A summation curve tabulated at equal time intervals. See "Summation curve".

Thiessen weight factor: A percentage factor which expresses the portion of rainfall at a particular rain gage which applies to a particular drainage basin. Computation of the factor is based on the Thiessen polygon method.

Definition of Terms--Continued

Mergence point, base flow: The point on the hydrograph at which all surface runoff has ceased and beyond which all flow is base flow. The expression is used in this report to define the point at which base flow recession curves should be merged with storm hydrographs for the purpose of combining the two.

Time (T), in hours: The number of hours measured from the beginning of storm runoff.

Time-to-peak (Tp), in hours: The time measured from the center of mass of rainfall excess to the resulting time of maximum instantaneous discharge (peak discharge).

Unit duration (d), in hours: The time during which rainfall excess occurs to produce a unit hydrograph. Sometimes referred to as unit time.

Unit hydrograph: A hydrograph of storm runoff as it would occur from one inch of rainfall excess uniformly distributed within one unit duration and uniformly distributed over the basin.

Unit time: See "Unit duration".

Symbols

A, drainage area, in square miles: See "drainage area".

d, unit duration, in hours: See "Unit duration".

L, basin length, in miles: See "Length, basin".

L_{ca}, basin mean length, in miles: See "Length, basin mean".

Q, discharge, in cubic feet per second (cfs): See "discharge".

ΣQ, runoff, in cfs intervals: See "Runoff".

R, runoff, in inches: See "Runoff".

T, time, in hours: See "Time".

T_L, adjusted lag time, in hours: See "Lag time, adjusted".

Tp, time-to-peak, in hours: See "Time-to-peak".

Δt, computation interval, in hours: See "Computation interval".