Flood Hydrology Manual

A Water Resources Technical Publication

U. S. DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

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by
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Surface Water Branch
Earth Sciences Division

FIRST EDITION 1989

United States Department of the Interior
Bureau of Reclamation
Denver Office

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

UNITED STATES GOVERNMENT PRINTING OFFICE
DENVER: 1992

For sale by the Superintendent of Documents, U.S. Government Printing Office,
Washington, DC 20402, and the Bureau of Reclamation,
Denver Offie, Denver Federal Center,
P.O. Box 25007,
Denver, Colorado 80225, Attention: D-7923A
024-003-00170-8.

PREFACE

The need for a comprehensive *Flood Hydrology Manual* for the Bureau of Reclamation has become apparent as increasing numbers of civil engineers are called upon to conduct flood hydrology studies for new and existing Bureau dams. This is particularly important because of the increasing emphasis that has been placed on dam safety nationwide. In general, these engineers possess varying backgrounds and levels of experience in the specialty area of flood hydrology. Accordingly, the primary purpose of this manual is to provide the necessary background, relationships, criteria, and procedures to allow the engineer to conduct satisfactory flood hydrology studies. As a result, these studies should reflect greater consistency and reliability of results for most of the drainage basins encountered in Bureau projects as well as those for other water resource construction agencies. These relationships, criteria, and procedures are based on detailed analyses of hydrologic and meteorologic data and studies of observed flood and severe rainfall events that have accumulated over the years.

The information contained in this manual reflects the methodologies currently used by the Bureau in performing flood hydrology studies. These methodologies have been proven to provide satisfactory results for use in the planning, design, construction, and operation of the Bureau's water control facilities. However, it would be inappropriate to infer that these methods are the final solutions to hydrologic problems because other methods are constantly evolving. It can reasonably be expected that, as additional data are collected and as new and more advanced techniques emerge, these methodologies will be improved upon and modified with the passage of time. It is also expected that such modifications and improvements will be fully incorporated into future editions of this manual.

The Bureau of Reclamation is primarily responsible for the development of water resources in the 17 Western States to meet agriculture, municipal and industrial, power, recreation, and environmental water supply requirements. In addition, with the recent emphasis on dam safety, the Bureau has been assigned the responsibility for safety of dams studies for Department of the Interior dams throughout the nation.

The Bureau has no authority, as legislated by Congress, in the area of flood control except for a few unique, specifically authorized projects. Flood control is, and has been for the past 50 years, within the purview of the United States Army Corps of Engineers. Therefore, the flood hydrology discipline, as applied to Bureau activities, deals with two principal areas: (1) determination of the upper limit or probable maximum flood potential at a damsite so that dams whose failure would result in loss of human life or widespread property damage can be designed to safely accommodate this flood without failure, and (2) determination of more commonly occurring floods for use in the design of diversion dams; very low hazard storage dams; cross drainage facilities for the extensive canals, aqueducts, and roads associated with the delivery of water to users; and for diversion of flood waters that may occur during the construction of dams. It should be noted that the technical hydrologic procedures used in these two principal areas are essentially the same as those used in flood hydrology studies that support the planning, design, construction, and operation of flood control projects.

Recognizing the Congressionally authorized function of the Bureau, this manual concentrates on three major technical aspects of flood hydrology: (1) hydrometeorology related to probable maximum precipitation determinations, (2) probable maximum flood hydrograph determinations, and (3) statistics and probabilities relating to the magnitude and frequency of flood flows. Other important, but essentially nontechnical, aspects related to flood hydrology studies are also treated.

Chapter 1, "Background and Historical Perspective," provides a brief historical perspective of the flood hydrology discipline. Included in this discussion is a general overview of procedures and philosophies that have evolved and been applied over the years, from those used in the development of early water control works to the present.

Chapter 2, "Basic Hydrologic and Meteorologic Data," provides a discussion of the basic hydrologic and meteorologic data that are available for analysis in the development of criteria for upper limit or probable maximum precipitation estimates, methods of determining rainfall-runoff relationships used in deriving probable maximum flood hydrographs, and in generating flood probability estimates. The methods and equipment currently used for collecting these data are briefly discussed to provide the reader with a general background on the mechanics involved. The important consideration of the reliability and accuracy of these data is included in subsequent chapters dealing with the specific uses of these data to provide the reader with an appreciation of the resulting accuracies of flood magnitude and frequency estimates. This chapter concludes with a more detailed description on assessing a drainage basin's physical features as they impact the rainfall-runoff phenomena through field reconnaissances.

Chapter 3, "Hydrometeorology," begins with a general discussion on the theories underlying basic atmospheric processes. Emphasis is placed on those processes that are important in the development of regionalized or generalized criteria used in estimating probable maximum precipitation amounts that are, as shown in subsequent chapters, necessary in determining probable maximum flood hydrographs. This chapter then describes the procedures used in the analysis of observed events that lead to the generation of criteria used for developing estimates of probable maximum precipitation and storm levels throughout the conterminous United States. The chapter concludes with remarks on continuing Federal interagency activities leading to the development of hydrometeorological criteria by the Bureau of Reclamation, National Weather Service, Corps of Engineers, and the Soil Conservation Service.

Chapter 4, "Flood Hydrograph Determinations," presents the theory and procedures currently used by the Bureau in converting rainfall over a drainage basin into a hydrograph of flood runoff. The hydrologic cycle is discussed as it pertains to extreme runoff phenomena up to and including the probable maximum flood. The process of infiltration losses into the soil and their effect on the runoff hydrograph is presented in more detail. Discussion on both natural and developed basins is included because development has been shown to considerably modify the hydrograph resulting from a given amount of precipitation. The chapter concludes with an example where a hydrograph representing the probable maximum flood event is generated for a relatively complex basin.

Chapter 5, "Flood Routings Through Reservoirs and River Channels," provides a discussion on procedures used by the Bureau in accomplishing the channel routing and combining of flood hydrographs. These procedures are needed when studying large basins that have been divided into subbasins or include one or more reservoirs. Procedures for routing flood hydrographs through reservoirs are also included.

Chapter 6, "Envelope Curves of Recorded Flood Discharges," provides a discussion on the purpose and procedures used to develop envelope curves of experienced flood discharges, both peak and volume. These curves provide a check on the adequacy of probable maximum flood hydrograph estimates. Proper procedures for segregating the data by region, season, storm type, and topography so that curves represent homegeneous hydrologic and meteorologic conditions are discussed in detail.

Chapter 7, "Statistics and Probabilities," provides a comprehensive presentation of an extremely important area of flood hydrology. The chapter includes a discussion of the basic concepts encountered in the collection and statistical analysis of streamflow data with emphasis on theoretical frequency distributions. Examples of these distributions are provided and their use in flood probability analyses is discussed. The chapter then discusses the current standard methodologies used by Federal water resources development agencies in determining discharge-probability relationships for basins in the United States. Considerable attention is focused on the uncertainties and relative reliabilities inherent in such relationships.

Chapter 8, "Flood Study and Field Reconnaissance Reports," is the final chapter of the manual, and discusses the importance of report preparation in documenting the basis and results of a flood hydrology study as well as the field reconnaissance report prepared during the initial phase of the study. The recommended content of such a report is also discussed.

This manual was written by Arthur G. Cudworth, Jr., (retired) former Head, Flood Section, Surface Water Branch, at the Bureau's Denver Office, with considerable assistance from Louis C. Schreiner and William L. Lane who were responsible for writing chapters 3 and 7, respectively. The authors are indebted to David L. Sveum, Kenneth L. Bullard, and John Dooley, all of the Denver Office, for their critical review of the manuscript and their valuable comments and suggestions. The authors are also grateful for the administrative support and direction of M. Leon Hyatt, Chief, Earth Sciences Division; Robert K. Lanky, Manager of Planning Services; and Terry P. Lynott, Assistant Commissioner for Resources Management, in bringing the preparation of this manual to a successful conclusion. In addition, we would be remiss if we did not extend our sincere appreciation to Diane C. Nielsen and Monica A. Galvan for their patience, perseverance, and untiring effort in typing the many draft versions and revisions.

Preparation of the manual for publication was supervised by the Editor, Ronald D. Mohr, under the general direction of Ronald E. McGregor, Head, Planning and Editing Section.

The Bureau of Reclamation expresses appreciation to the organizations who have permitted the use of their material in this text. There are occasional references to proprietary materials or products in this publication. These references are not to be construed in any way as an endorsement because the Bureau does not endorse proprietary products or processes of manufacturers or the services of commercial firms.

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Chapter 1
BACKGROUND AND HISTORICAL PERSPECTIVE

1.1 Background

Archeological evidence indicates that man has been attempting to regulate the flow of water in streams and rivers for beneficial purposes for about 6,000 years. This evidence generally consists of the remnants of dams and irrigation systems. Robert B. Jansen, former Assistant Commissioner of the Bureau of Reclamation, in his excellent publication *Dams and Public Safety* [1][1], states that, "History does not record exactly when irrigation systems were first constructed. Study of ancient China, India, Iran, and Egypt does reveal that such work in these lands was begun thousands of years ago, and provided lifelines on which their civilizations depended. Menes, the first Pharaoh of Egypt, ordered irrigation works to draw from the River Nile. In China, construction of impressive dams was accomplished on the Min River for flood control and diversion of water to nearby farmlands. The sacred books of India cite the very early operation of dams, channels, and wells; evidence that this land may have been the birthplace of the art. The Persians of ancient times recognized the importance of irrigation to the sustenance of civilization. By excavating underground water tunnel and gallery systems (quanats) and by constructing many dams, they accomplished projects which rank among the greatest in history. In the ruins at Sialak, near Kashan, are to be seen traces of irrigation channels which are considered to be as much as 6,000 years old, suggesting that irrigation was practiced there from very early times, even before the arrival of the Aryans in the land now known as Iran." Jansen goes on to recount a number of dam failures associated with these ancient works, many of which were apparently the result of an inability to safely pass or accommodate floods that their tributary watersheds had produced.

Ven Te Chow [2], in his 1962 publication *Handbook of Applied Hydrology*, termed the period dating from these ancient times up until about A.D. 1400 as the "Period of Speculations." During this period, many noted philosophers speculated, often erroneously, about the nature of the hydrologic cycle. However, due to the number of water control works constructed, these philosophers gained a certain measure of practical knowledge. Unfortunately, this knowledge and the lack of scientific rainfall and runoff measurements was insufficient to enable the designers of that time to estimate, in quantified terms, the flood potential at their damsites. Generally, this led to underdesign and ultimate failure of their constructed works. It was not until the 17th century, the period that Chow termed the "Period of Measurements," that the science of hydrology commenced. The first measurements of rainfall, evaporation,

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[1]Numbers in brackets refer to entries in the Bibliography.

and river discharges were made in Central and South Central Europe. Measurements of river flow were accelerated in the 19th century with the development of measuring instruments such as the Price current meter, which remains in use today. The 19th century marked the installation of the first stream-gauging station by the USGS (U.S. Geological Survey) [3]. This station, established on the Rio Grande River at Embudo, New Mexico by John Wesley Powell in December 1888, served as the forerunner to systematic stream gauging, or "discharge data acquisition," as it is currently known. The Embudo Camp, using the Embudo gauge, served as a training facility for instruction in the use of instruments and the methods used in the relatively undeveloped art of stream gauging. The establishment of a systematic stream gauging program by the USGS provided the needed data base for later (20th century) development of both physical and statistical flood hydrology procedures presently in use by the major Federal water resource construction agencies.

1.2 Historical Procedures

In the late 1800's and early 1900's, statistical procedures were used by some in evaluating the flood potential of basins where dam construction was planned. In other cases, designers based their dam designs on the historic flood of record raised to a higher value by applying a factor. These factors were at best quite arbitrary. Another approach used was the application of a number of empirical formulas generally relating peak discharge to drainage area, a method proposed during the 19th century. Each of these three approaches was used with apparently satisfactory results until a number of flood events occurred that could not have been reasonably predicted by application of any of these approaches.

Four examples of occurrences where floods were much greater than previous maximum recorded floods are graphically presented on figures 1-1 through 1-4. These examples illustrate the extreme variability of flood flows and the potential consequences of relying on historic peak discharges as a basis for design of high hazard water control structures. These four examples and one other are discussed in the following paragraphs.

(a) Arkansas River.—The Arkansas River at Pueblo, Colorado rises near the Continental Divide and drains areas exceeding 14,000 feet in elevation. The river leaves the mountains near Canon City, Colorado, and flows easterly across a high plains region passing through Pueblo. Systematic stream gauging on the Arkansas near Pueblo was initiated with the establishment of the Pueblo gauge in 1885, with continuous daily records starting in 1895. From sometime in 1885 to June 3, 1921, the annual maximum peak discharge recorded was 30,000 cubic feet per second on May 21, 1901. Each annual maximum flow was the result of the spring snowmelt runoff from the higher elevations in the basin. On

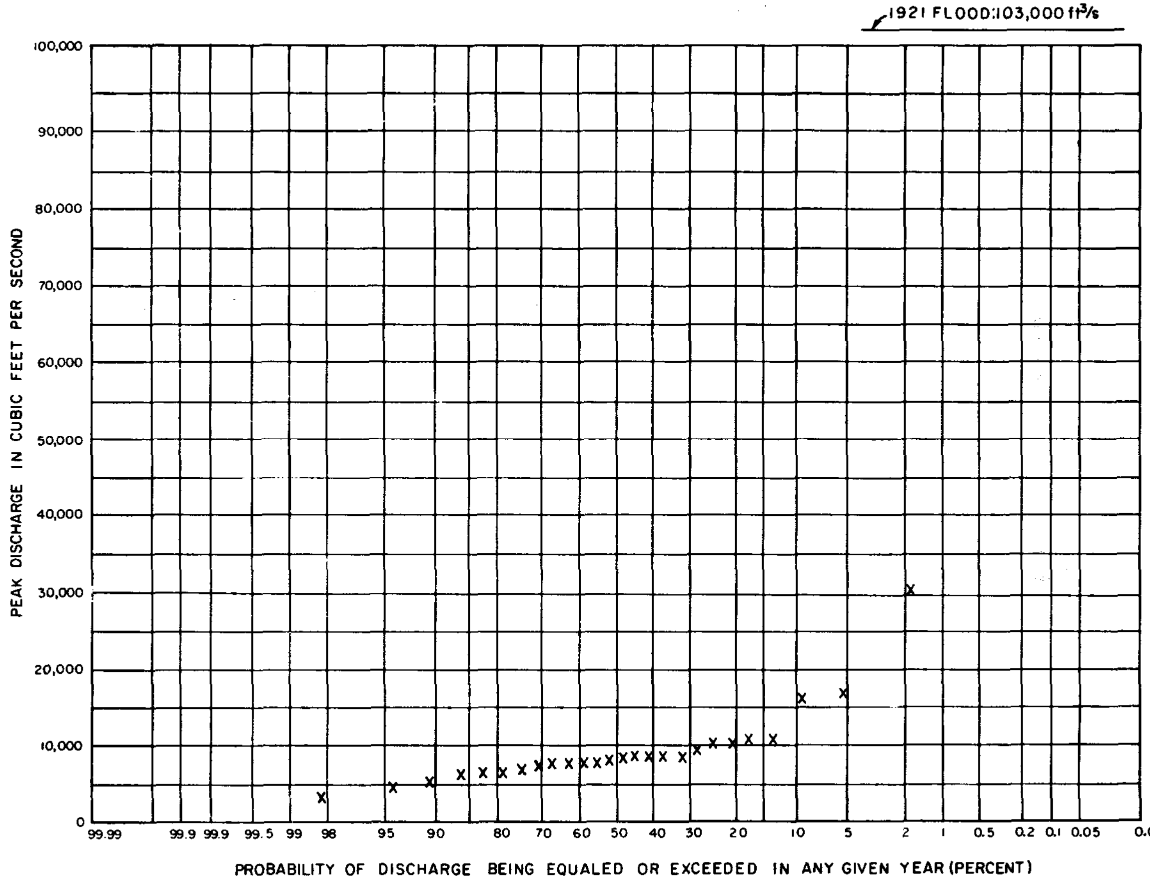


Figure 1-1.—Discharge-probability relationship, Arkansas River near Pueblo, Colorado, 1895-1920. 103-D-1901.

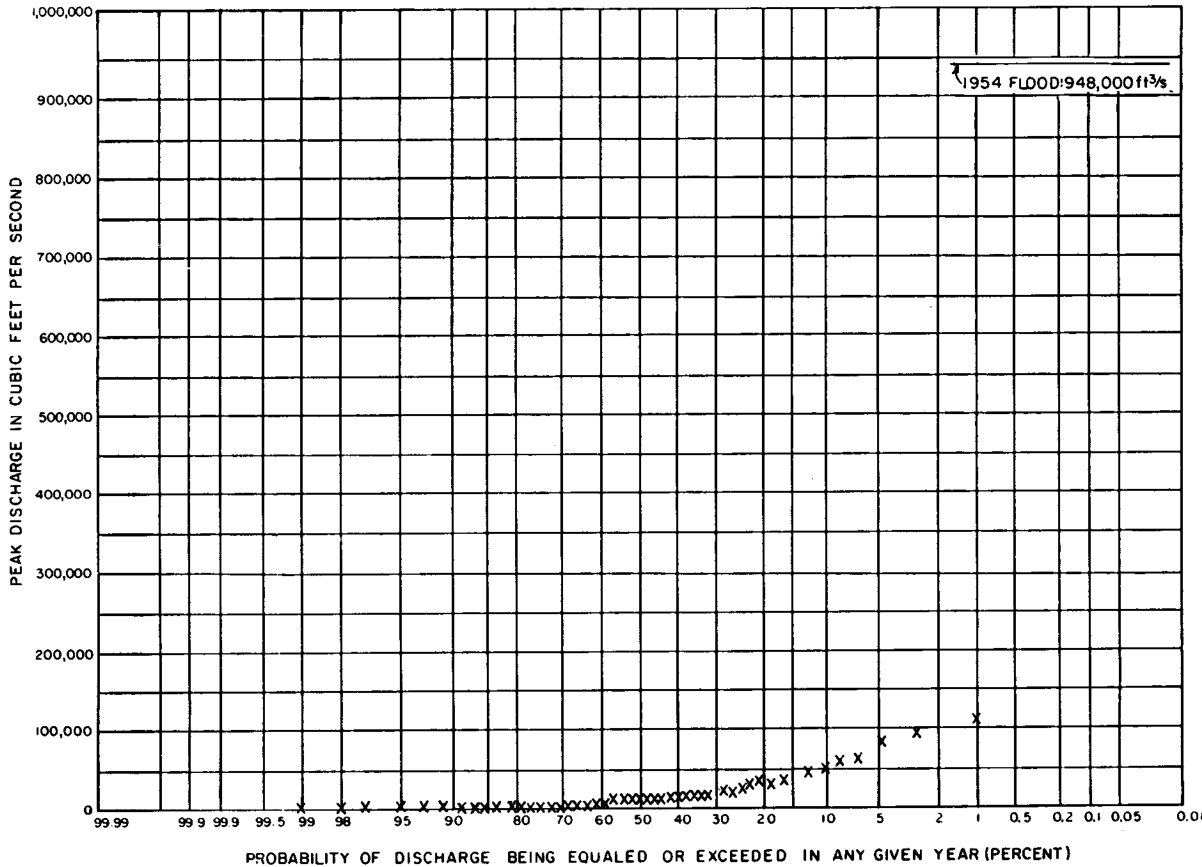


Figure 1-2.—Discharge-probability relationship, Pecos River near Comstock, Texas, 1901-53. 103-D-1902.

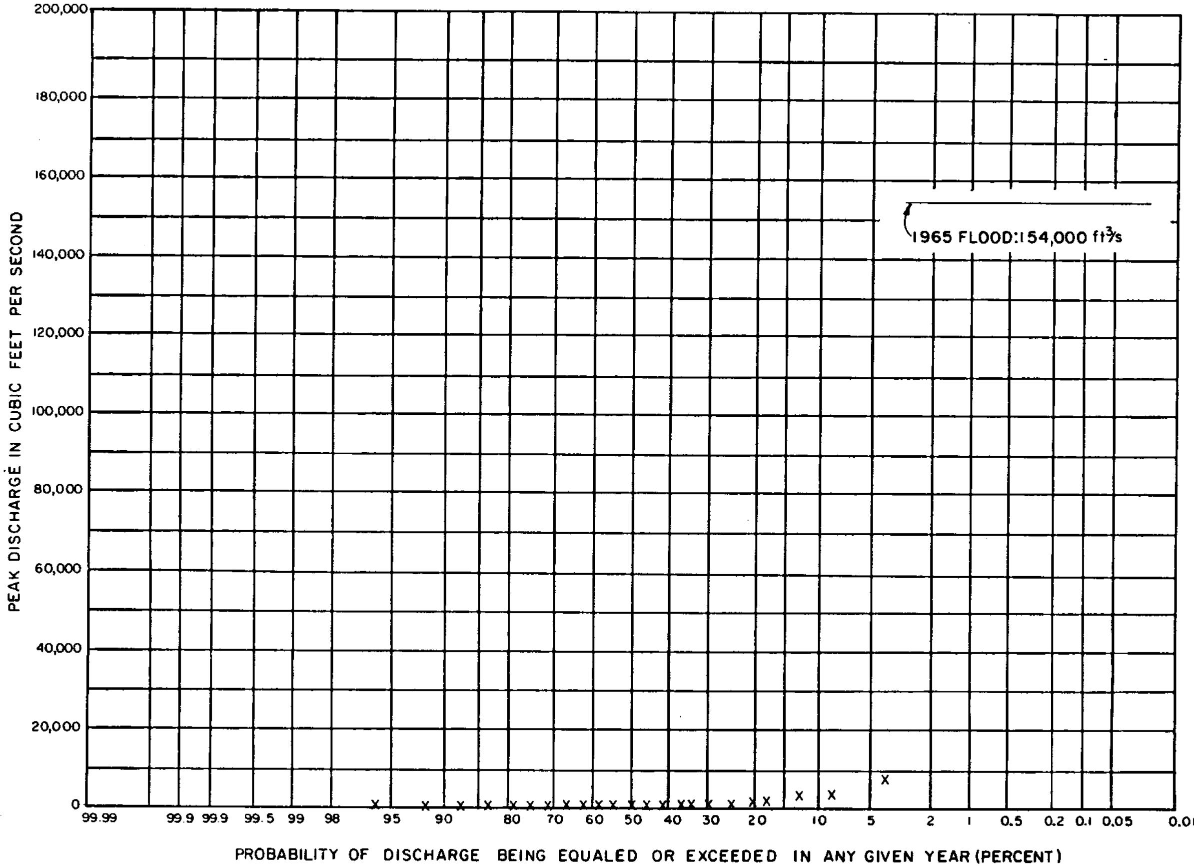


Figure 1-3.—Discharge-probability relationship, Plum Creek near Louviers, Colorado, 1942-64. 103-D-1903.

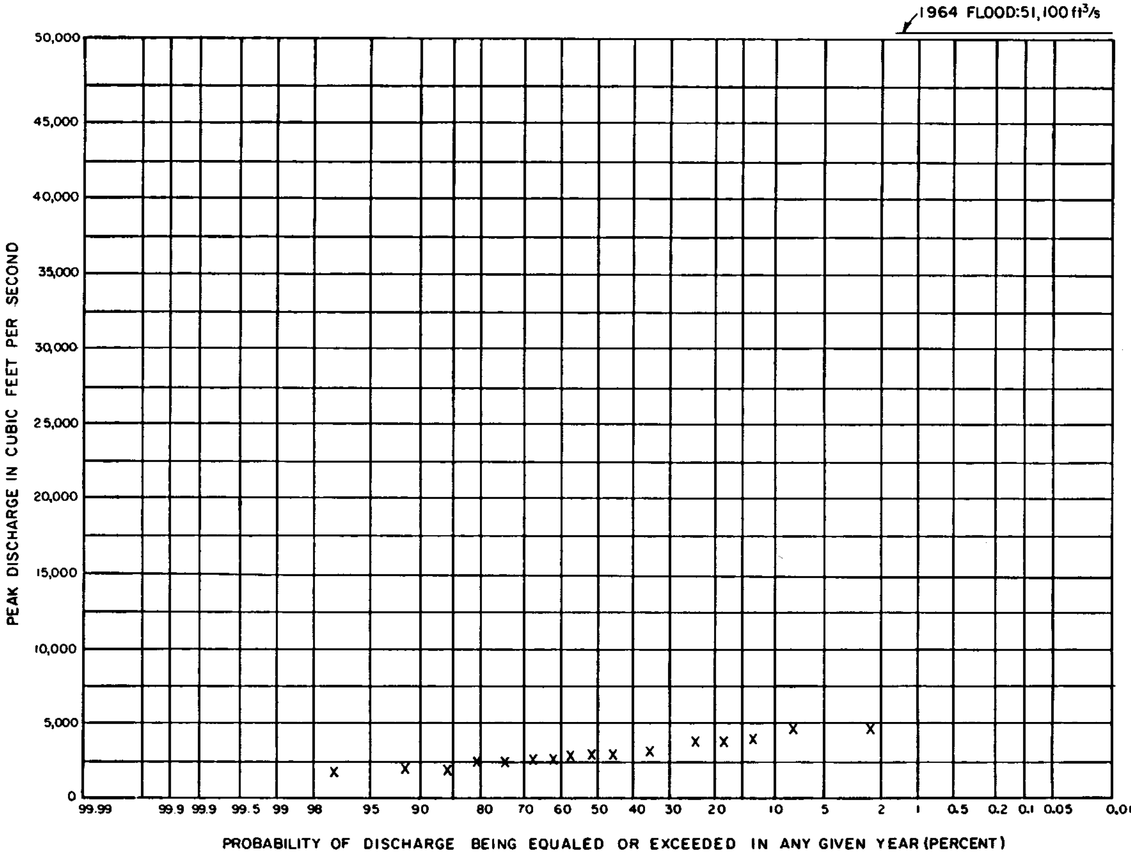


Figure 1-4.—Discharge-probability relationship, Nork Fork of Sun River near Augusta, Montana, 1946-63. 103-D-1904.

June 4, 1921, a large Great Plains type thunderstorm centered over the high plains region immediately upstream from Pueblo. This extreme event, with rainfall amounts of over 10 inches within a 6-hour period, produced a peak discharge at the Pueblo gauge of 103,000 cubic feet per second. This discharge represents a threefold increase over the previously experienced recorded maximum shown on figure 1-1.

(b) Pecos River.—A stream gauge was installed on the Pecos River near Comstock, Texas, in 1901, and measured runoff from a 35,162-square mile area of the Pecos River Drainage Basin. During the first 53 years the gauge was in operation, the highest annual peak flow recorded was 116,000 cubic feet per second in 1932. In June 1954, a major storm event centered over the lower 2,500 square miles of the basin immediately above the Comstock gauge. This severe rainfall event produced a measured peak discharge of 948,000 cubic feet per second. As shown on figure 1.2, this event was almost nine times higher than any recorded event in the previous 53 years.

(c) Plum Creek.—One of the most significant examples in terms of exceedance of previously experienced discharges was the June 16, 1965 flood on Plum Creek, Colorado. A severe storm similar in type to the June 1921 event in Pueblo, Colorado centered over this 302-square mile drainage area in the foothills of the Colorado Rocky Mountains. The resulting discharge from this storm was measured at 154,000 cubic feet per second. Systematic measurements of streamflow at the Plum Creek near Louviers gauge since 1942, yielded a maximum peak discharge of 7,700 cubic feet per second. As shown on figure 1-3, the 1965 event was about 22 times larger than what was experienced in the previous 23 years that the gauge was in operation.

(d) Sun River.—Such phenomena are not limited to the high plain regions. In the mountainous region of Montana in early June 1964, a severe storm on the Sun River produced a flood event of such extreme magnitude that it overtopped the Bureau's concrete-arch Gibson Dam. Streamflow records accumulated at the USGS stream gauge on the North Fork of the Sun River near Augusta, Montana (fig. 1-4) had recorded an annual maximum discharge of about 4,800 cubic feet per second for the period from the time the gauge was installed in 1946 through 1963. This event produced a flood with a peak discharge of 51,100 cubic feet per second, over 10 times the previous maximum peak recorded discharge.

(e) Ware River.—As a final example, and there are many more, consider the Ware River at Gibbs Crossing, Massachusetts. A stream gauge was established at this location in August 1912. From 1912 to March 1936, the maximum recorded peak flow was measured at 2,960 cubic feet per second in 1933. In March 1936, a flood occurred that had a recorded peak of 11,200 cubic feet per second, four times larger than the previous maximum. Only 2 years later in September 1938, as a result of extreme rainfall that took place during a hurricane, a peak discharge of 22,700 cubic feet per second was recorded. This latter flood event had a peak twice as large the preceding record event and over seven times the next largest event.

It can readily be seen from these few examples that engineers, basing their dam and spillway design on either factoring up a recorded maximum flow or by applying statistical procedures to estimate a given flood probability, could greatly underestimate the flood potential of a drainage basin and develop an inadequate hydraulic design. In many instances, this has proved to be the case. A. O. Babb, in his 1968 Bureau publication *Catalog of Dam Disasters, Failures, and Accidents* [4], catalogs 55 failures of private and public dams in the United States due to overtopping during the period from 1889, when the well known Johnstown, Pennsylvania disaster occurred, to the mid 1930's. This series of events probably led the dam engineering community, in the mid 1930's, to seek ways to more reliably estimate the upper limit flood potential of a drainage basin.

1.3 Hydrometeorologic Approach

The 1930's saw the basic hydrologic and meteorologic tools become available for what became known as the "hydrometeorologic approach" to upper limit or PMF (probable maximum flood) hydrograph development. In 1932, Leroy K. Sherman [5] proposed the unit hydrograph theory which, with modifications, is still being used by the Bureau and other major Federal water resource development agencies. This theory, discussed in more detail in chapter 4, provided the basis, or model, for converting rainfall excess on a drainage basin to a hydrograph representing surface runoff. A year later in 1933, Robert E. Horton [6] proposed his infiltration theory, which provided the analyst with the capability to account for the amount of rainfall, falling on a drainage basin, lost by infiltration into the ground and therefore not available for surface runoff. In 1933, the only significant part of the procedure missing was a technique for estimating the upper limit rainfall amounts that could be used to compute the PMF hydrograph. In 1936, Merrill Bernard [7] of the NWS (National Weather Service) and Gail Hathaway of COE (Corps of Engineers) reached a cooperative agreement whereby NWS would establish and COE would fund a specialized group of professional meteorologists to study severe meteorologic phenomena and develop criteria that could be applied by the hydrologic engineer to determine upper level or PMP (probable maximum precipitation) values. This group of meteorologists became what is presently known as the Hydrometeorological Branch in the Office of Hydrology at NWS. This branch has published a series of reports, generally referred to as the HMR (Hydrometeorological Report) series, that provide definitive criteria for developing PMP estimates.

Beginning in 1980, the activities of the Hydrometeorological Branch have predominately been steered by an informal committee comprised of representatives of the Bureau of Reclamation, Corps of Engineers, Soil Conservation service, and the National Weather Service [8]. The primary objective of this committee has been to assure that consistent, reliable, and reasonable PMP criteria are available for use by the dam engineering community for the conterminous United States. This objective has been achieved with the publication of HMR 55A. The most up-to-date publications in the report series, as shown with their regions of coverage, are shown on figure 3-7 in chapter 3. The derivation of these criteria is discussed in general terms in chapter 3, and a detailed derivation is presented in each individual report.

The present practice and policy of the Bureau is to use these reports for determining PMP values for virtually all PMF studies. Exceptions are encountered when the drainage basins being studied are larger in areal extent than that covered by the reports. This practice departs from past Bureau policy where individual hydrometeorological studies were conducted for individual basins tributary to dams. Review of these studies under the Bureau's safety of dams program indicated that these past practices yielded results that were frequently inconsistent within meteorologically homogeneous regions, did not systematically account for all available historically observed storms suitable for analysis, and did not always reflect the most efficient meteorologic processes. To overcome these deficiencies, it became Bureau policy to adopt the HMR series for estimating PMP and, in turn, developing PMF hydrographs.

The unit hydrograph technique and Horton's infiltration theory have been the basic tools used by the Bureau for many years to convert rainfall to flood runoff. The document *Unit-Graph Procedures* [9] was published by the Bureau in November 1952, and embodies the basic principles advanced by Sherman 20 years earlier. The techniques outlined in this document were modified to incorporate work conducted by hydrologic engineers in the Bureau and in the Los Angeles District and South Pacific Division offices of COE in the 1960's and early 1970's [10]. As will be discussed in detail in chapter 4, the principal modification to the document was that of introduction of a factor into the general unit hydrograph lag time equation that varies as a function of a drainage basin's hydraulic efficiency characteristics. The introduction of this factor was generally considered to improve the reliability of synthetic unit hydrographs derived for ungauged drainage basins.

Even though statistical or probabilistic methods for determining upper limit flood hydrographs have been discarded in favor of the deterministic or hydrometeorological method, considerable work in flood hydrology relies on data developed using the former methods. Statistical analyses are generally required to estimate flood discharges used in assessing cross drainage requirements, hydraulic and structural design of diversion dams, establishing crest elevations of auxiliary and fuse plug spillways, river diversion requirements during dam construction, and for determining economic benefits for flood control projects. As previously mentioned, the determination of flood control benefits and the associated statistical hydrologic analysis lie within the province of COE because of their legislated authority. Statistical analyses associated with cross drainage design and river diversions are a problem common to all water resources development entities. Prior to 1967, these entities used several different technical approaches to determine discharge probability relationships. On several occasions, two or more entities conducted discharge probability studies for the same basin, with results that often varied considerably. At that time, this situation led the National Water Resources Council to implement a study group comprised of representatives of each Federal water resource development agency to develop guidelines, using a single technique, that would be applied by all agencies in the development of discharge-probability relationships. The goal of this effort was to achieve the greatest degree of consistency possible in probabilistic studies conducted by these agencies. The result of this 1967 effort was the publication of *Water Resource Council Bulletin 15* [11], which was placed in use by all agencies. Subsequently, as problems with Bulletin 15 criteria became apparent through application, Bulletins 17 [12], 17A [13], and 17B [14] were successively published, each being essentially an improved revision of its predessor. Currently, Bulletin 17B provides the basic probabilistic criteria being used by the Federal establishment. These criteria and their development are discussed in chapter 7.

Hopefully, this chapter has provided the reader with some general background information as to where the specialty area of flood hydrology has been and where it is today. The following chapters of this manual will provide definitive information on current procedures used by the Bureau of Reclamation in estimating PMP values, deriving unit hydrographs, assigning appropriate infiltration loss rates, computing PMF hydrographs, routing flood hydrographs through reservoirs and river channels, and conducting statistical analyses of streamflow data.

Chapter 2
BASIC HYDROLOGIC AND METEOROLOGIC DATA

2.1 General Considerations

The most fundamental part of any flood hydrology analysis is the compilation and analysis of hydrologic and meteorologic data accumulated during and after severe flood events. As will be seen in later chapters, these data are required in the development of criteria by hydrometeorologists for making PMP estimates, development of unit hydrograph and infiltration parameters necessary to determine the rainfall-runoff relationships for both gauged and ungauged basins, and for preparing discharge-probability relationships. Hydrologic data include records of flood runoff measured at continuous recording streamflow gauges, crest stage streamflow gauges, indirect peak discharge measurements based on flood marks at locations where there are no stream gauges, and reservoir operation records from which inflow hydrographs may be determined based on outflow and change of storage relationships. Meteorologic data include precipitation, temperature, dewpoints, and wind records collected at "official" NWS first and second order climatological stations; snow surveys conducted by Federal, State, and local agencies in areas susceptible to significant snowmelt runoffs; and data from supplemental precipitation surveys, commonly referred to as "bucket surveys", conducted immediately after the occurrence of severe storm and flood events. The purpose of the latter is to provide additional data which, when used with data collected at the official NWS stations, provide fairly definitive information on the areal extent, timing, and intensity of a storm. Other vitally important data include data related to watershed characteristics such as topography, amount and type of vegetation, geologic setting, drainage network development, and degree of development.

2.2 Hydrologic Data

(a) Recorded Streamflow Data.—These data are collected primarily by the USGS at continuous recording streamflow gauging stations. A comprehensive discussion on the installation equipment and operation of these stations is presented in USGS Water Supply Paper 888, *Stream-Gaging Procedure* [15][1]. Briefly, the stations are equipped with devices that sense and record the variation of the river stage above an arbitrary datum. There are two primary types of sensing and recording devices currently in use at these stations. The first type is actuated by a float installed in a well located below the gauge house. The well is connected

[1]Numbers in brackets refer to entries in the Bibliography.

to the river using pipe intakes that allow the water level in the well to maintain the same elevation as the water level or stage in the river. The float is connected to a recording device by a counter-weighted cable. As the river stage rises or falls, the float in the well does likewise, causing the counter-weighted cable to move upward or downward. The cable is connected to a drive wheel on a recording device, generally a Stevens type A-35 recorder, that causes a pen in the device to move back and forth with the changing river stage. The pen, usually filled with waterproof ink, records the stage on a continuously moving recorder chart that moves at right angles to the movement of the pen, and is driven by a clock-actuated device. The clock drive advances the paper chart at a fixed rate so that, in most cases, 1 inch on the recorder chart equals 12 hours. As a result, the water-level recording device produces a continuous trace on the chart of the variation of river stage over time.

The second type of device is called a "bubbler gauge." Rather than sensing the water surface level by using a well and float, the bubbler gauge feeds nitrogen gas under pressure through a tube to an orifice located at or near the bottom, or thalweg of the river channel. Because pressure must be exerted on the gas to overcome the water pressure or head at the orifice, it is possible to measure the variations in pressure and relate them to variations in the water surface level, or stage, in the river as the pressure in the tube corresponds to the head on the orifice. The variation in pressure is then sensed by a mercury servomanometer which, in turn, drives a recording device, usually either a Stevens A-35 or a Fisher-Porter punch-tape unit. The A-35 device produces a continuous trace of the river stage whereas the punch-tape unit only records the stage at predetermined time intervals. The time interval set for a punch-tape unit is of importance in flood hydrograph analyses, particularly for smaller drainage basins. If the time interval between readings is too long, it is possible to miss recording the peak of a runoff event, which would render the record unusable in observed flood hydrograph analyses, as discussed in chapter 4. To overcome this problem, many bubbler-gauge installations include both a punch-tape unit set at a relatively long time interval and a continuous recording device such as the Stevens A-35. The primary advantage of the punch-tape unit is that the data recorded on the paper tape can be extracted by automated means, whereas the continuous recording device requires manual reduction of the data.

The recorded river stage data, whether accumulated by a continuous recording device or a punch-tape unit, must be converted to discharge values for use in the hydrologic analyses. This is accomplished by developing a rating curve for the stream gauging site that relates discharge to river stage. Discharge measurements, over the maximum range possible, are usually made using a current meter that measures the velocity of flow, and either staffs or calibrated cables that are used to measure depths of flow across the cross section of the channel at the measuring section at or near the gauging station. In general principle, the velocity of flow is measured at several locations across the channel resulting in an average velocity for the section. The staff or cable readings for depth of flow are taken at each point that the velocity is measured. Horizontal measurements are taken between staff readings. The depth reading times the sum of one-half the distance between adjacent staff readings yields the cross-sectional area that may be applied to the velocity to obtain the discharge for that part of the total cross section. The total discharge of the section at the particular stage is then found by adding the discharge determined for each part of the total section. This general procedure is repeated for various river stages, and a plot of discharge versus river stage is then constructed which is the rating curve for the station. In actual practice, the procedure is somewhat more complicated than described here, and a detailed discussion on the procedure is beyond the scope of this chapter. If interested, the reader is encouraged to study the USGS Water Supply Paper No. 888, *Stream-Gaging Procedure—A Manual Describing Methods and Practices of the Geological Survey,* dated 1962 [15].

The hydrologic engineer should, however, be aware of the relative accuracies inherent in these data. The streamflow records published by the USGS will provide, for each station, a statement as to whether the records are excellent, good, or fair. In many instances, the records will be rated at one level up to a given discharge and then rated lower at higher discharges. This is generally due to the inability to obtain accurate velocity measurements at higher flows, or to having a hydrographer present to measure the flow when the peak discharge occurs. As a result, the rating curves have to be extrapolated beyond the measured data. A rating of excellent implies a velocity measurement within 2 percent accuracy, a rating of good implies an accuracy of the velocity measurement within 5 percent, and a fair rating implies an accuracy within 8 percent. If the upper end of the rating curve is based on indirect measurements, the accuracies of the resulting discharge data may be less; this is discussed later in section 2.2(b).

The streamflow records are compiled and published by the USGS in a series of "Water Supply Papers" that are generally available in each Bureau office, local USGS offices, and in major city libraries. These publications present streamflow in terms of average daily flow for the period of time that the stream gauge has been in operation. The data are suitable for developing both peak- and volume-probability relationships as discussed in chapter 7; however, the data are of limited value for observed flood hydrograph analyses (ch. 4) in all but very large drainage basins. Accordingly, the data are rarely used in hydrograph analyses. Average daily flow values are developed from recorder charts that provide a continuous record of river stage versus time at each gauging site. These charts can provide valuable information for the flood hydrograph analyses. River stage is shown on the recorder chart as an elevation, in feet, above some arbitrary datum. Copies of these charts can be obtained from the Water Resource Division in the local offices of the USGS along with the rating curve for each gauging station. A hydrograph representing the discharge in cubic feet per second for a particular location can then be developed by reading river stage values from the recorder chart at selected time intervals, and then converting those values to discharge using the rating curve for that station. The time interval selected should be sufficiently short to properly define the recorded hydrograph, especially in the case of hydrographs with rapid rise and recession limbs. Proper hydrograph definition is important to gain an understanding of the runoff characteristics of these gauged basins. This point is treated in more detail in section 4.1 of chapter 4.

The previously mentioned Water Supply Papers also present the instantaneous peak discharge for each gauging station for each year that the station has been in operation. These data form the basis for developing the discharge envelope curves discussed in chapter 6, and the annual peak discharge-frequency relationships discussed in chapter 7. In addition, some peak flow values less than the maximum annual event but greater than a base flood flow level are presented, which are of use in developing partial duration discharge-probability relationships.

The process of abstracting the required data from the many volumes comprising the Water Supply Papers for use in the statistical analyses discussed in chapter 7 is tedious, laborious, and expensive in terms of manpower requirements. There is also a high potential for human error when performing this task manually. With the advent of current high-speed electronic computers and their large data storage capabilities, the USGS has provided for storage and retrieval of these data in a system known as WATSTORE (National Water Data Storage and Retrieval System). In addition to storing streamflow records for over 11,000 stream-gauging stations presently in operation, the system also stores data for about 20,000 stations that are no longer operating. The Bureau currently has access to this system in all of its project and regional offices, and in the Denver Office. As the procedures for accessing the system are modified from time to time, the new user should contact personnel of the Surface Water Branch, Flood Section, at the Denver Office for current accessing procedures.

(b) Indirect Discharge Measurements.—The costs associated with installing, operating, and maintaining continuous recording streamflow gauging stations and compiling and publishing the resulting data are rather high. In view of these high costs, there are relatively few continuous recording stream gauges in the United States considering the number and length of all the rivers and streams involved. To supplement the data acquired at these stations, indirect discharge measurements are taken at other locations following flood events.

Indirect discharge measurements are of considerable importance in flood hydrology in the development of envelope curves of experienced peak discharges (ch. 6), and in the development of peak discharge-probability relationships (ch. 7). In most cases, these indirect measurements determine only peak discharge and are therefore of only limited value in observed flood hydrograph analyses, as discussed in chapter 4. There are a few instances where an observer has marked flood heights in a permanent fashion at recorded time intervals during both the rise and fall of the water surface during a flood event. This provided a stage versus time relationship that was then converted to a flood discharge hydrograph using either an existing rating curve or a curve developed from data derived using slope-area procedures.

Also important to an adequate indirect measurement is the establishment of the high water marks attained during the flood event. One of the best methods of recording the maximum stage is by using a crest stage gauge. This type of gauge is currently installed and operated principally by State highway departments in many regions of the country. They are simple devices consisting of a length of 2-inch-diameter pipe mounted vertically on a post or bridge pier. The pipe is capped at each end with the lower part of the pipe perforated in the direction of flow to permit entry of water. A graduated rod is inside the pipe and granulated cork is placed at the bottom. Entry of water during flood flows causes the cork to rise and adhere to the rod at the maximum stage achieved during the event. This maximum stage is then related to discharge using a "rating curve," if one exists, or the discharge is determined from the maximum stage using the "slope-area" method of indirect peak discharge measurements, as described in USGS Water Supply Paper 888 [15].

Indirect discharge measurements are made as soon as practicable after flood events. The measurements are based on hydraulic computations using the flood water's maximum stage, as it can be ascertained, usually through a short section of the river or stream. The maximum stage may be in the form of physical evidence such as debris or scour along the banks or overflow area of a watercourse, at bridges along the stream, or may be actually recorded as in the case of flow over a dam's spillway by the reservoir's water-level recorder. Chow [16] cites five categories of this type of measurement including (1) measurements taken along channel reaches, (2) those taken at contracted openings such as bridges, (3) readings taken where flow passes through culverts, (4) where flow takes place over what may be considered a broad-crested wier such as a highway embankment, and (5) where flow passes through a hydraulic structure such as a spillway or outlet works at a dam. Another category where indirect discharge measurements are used is called the "crest stage gauge," where high water stages are recorded.

The majority of indirect discharge measurements are made along channel reaches using the slope-area method [15,16] that uses Manning's equation, as described in section 5.4. This equation is used to determine the average velocity in the reach and then, having measured the cross-sectional areas along the reach, this average velocity is multiplied by the average cross-sectional areas to arrive at the discharge estimate. Care must be taken in selecting the reach to minimize the variation in cross-sectional areas and to be reasonably sure that steady flow occurred at the maximum stage reached during the flood event. Naturally, the selection of proper Manning's n values is critical to an adequate estimate, requiring some experience on the part of the analyst.

It should be noted that the accuracies of flow measurements using the indirect method are not as good as those of the systematic recording gauge. The USGS classifies as "good" those measurements within 10 percent of the true value, and "fair" for those within 15 percent.

2.3 Meteorologic Data

(a) Systematic Data Acquisition.—Systematic acquisition of precipitation data is accomplished primarily through the efforts of the NWS, who maintains a network of what are termed "first order" weather stations that are operated on a 24-hour basis by meteorologists. Each weather station in this network collects continuous precipitation, temperature, wind, and relative humidity data. A complete set of microfiche containing these systematic data is maintained by hydrometeorologists in the Flood Section at the Bureau's Denver Office for use in PMP and storm studies for which they are responsible. Also, the Flood Section maintains records for "second order" NWS stations and for data collected by NWS cooperative observers. A second order station is operated by meteorologists during the day, collecting the same data as a first order station. Cooperative observers collect 24-hour rainfall and maximum and minimum temperature data.

(b) Supplemental Meteorologic Data Acquisition or Bucket Surveys.—Hydrometeorologists require data on historical storms for use in preparing PMF hydrographs and developing operating criteria for performing flood routings through reservoirs. With the numerous recording rain gauges now operating, as compared with the past, relatively more complete data are being obtained for current storms. However, the network of precipitation stations is still far from sufficient to provide the necessary temporal and spatial data for detailed analyses of observed storm precipitation. It is therefore necessary and extremely important, following outstanding storms, to supplement the data obtained at "official" rain gauges with "unofficial" observations. These unofficial observations may be made by individuals, radio and TV stations, and city and county public works departments. These observations include measurements of precipitation caught in any type of open receptacle. As Bureau personnel may initiate or be called upon to participate with other interested entities in these important supplemental precipitation surveys, the following, rather detailed, guidance is provided relative to recognizing the need for, and developing procedures for, conducting these surveys. It must be reemphasized that this activity is of the highest importance in the area of hydrometeorology.

(1) Within the Bureau, the responsibility for recognizing particular storms as being outstanding and for collection of supplemental rainfall data lies with the regional and project offices in whose area the storm occurs. The occurrence of unusual precipitation events may be indicated by newspaper, radio, and television reports of precipitation and stream or river stages and from observations available at local weather service offices. However, these local information sources does not reduce the responsibility of the Bureau offices to be independently cognizant of the occurrence of severe events in their areas of responsibility. When such an event occurs, efforts should be made to coordinate the decision to conduct a bucket survey with hydrometeorologists of the Flood Section at the Bureau's Denver Office, as well as other interested agencies, primarily the COE, NWS, and SCS. State and local agencies that might have an interest should also be approached for possible involvement during these preliminary coordination activities. In the interest of preserving and maximizing the reliability of the data, *the data must be collected as quickly as possible* after the occurrence of the storm so that the decision on whether to conduct a survey can be made quickly. The plotting of available reports on precipitation amounts on a map of the general area will be helpful at this phase, as well as later in the survey. In particular, this plotting will later allow the survey team to concentrate their efforts in what appear to be data deficient areas. If the available data show the possibility of rainfall rates, in any part of the area for any duration, approaching or exceeding the maximum observed for such area and duration, a supplemental precipitation survey of the storm should be made. In general, reported rainfall equivalent to that of an actual or potential major flood-producing storm should be investigated.

(2) When the decision has been made that a particular storm merits investigation, detailed plans for the field survey should be coordinated with others that have indicated interest at the preliminary stage. Local offices of the Bureau of Reclamation in or near the storm area, the appropriate project or regional office of the Bureau, district office of COE, and appropriate regional offices of NWS, USGS, State engineers, and local agencies should be contacted to determine their capability in supporting the field survey and whether they desire to participate as survey team members. Plans should be adjusted to eliminate duplication of effort among the participants, and to ensure full coverage of the area over which the storm occurred. In general, it is best to accomplish the necessary coordination in person or by telephone because correspondence by mail has been found to involve unacceptable delays for storm survey work.

(3) It is difficult and not particularly desirable to give the survey team members hard and fast instructions on the routes they are to travel in

obtaining the rainfall data. Generally, it is considered best procedure for the team to first locate the center or centers of precipitation intensity and then obtain data along lines crossing those centers. To a large extent, the itineraries of team members will be governed by the distribution of population in the storm area and the location and condition of the roads and bridges. County highway maps have been found to be very useful in assigning areas to team members and in identifying the location of data points. The instructions should be specific enough to preclude the omission of significant areas and the overlapping of work areas. Emphasis must be placed on obtaining every bit of pertinent information available, particularly near the center of the storm. In planning the survey, it is helpful to inform local residents of the existence and purpose of the survey and the locations where team members may be reached. Extensive use of news media is desirable in this effort. Specific items for the survey team to follow are:

- Bureau of Reclamation Form HD39 is intended for use in field surveys of storm precipitation. The upper part of the form is devoted to the identification of the location of the observation, and should be carefully filled out before leaving the site. Figure 2-1 shows a sample of this form that has been completed in a proper manner.

- The team members' approach in interviewing residents should be businesslike, but pleasant and tactful, briefly explaining the purpose of the survey. For the purpose of interest, some of the information already obtained could be mentioned during the interview. The times of beginnings and endings of precipitation should be obtained from the interviews, if possible. If precipitation was discontinuous or if its character varied, the beginnings and endings of the separate occurrences, heavy precipitation, and snow or hail should be obtained and entered on the form. The observation of thunder, lightning, strong winds, or wind shifts should also be noted. All data may be of value regardless of whether the person interviewed has actually measured the precipitation. The NWS cooperative observers frequently do not enter much data on their forms other than the amount of precipitation; therefore, it is usually beneficial to interview them for that type of additional information.

- In obtaining measurements of precipitation, the most desirable open receptacles are level, straight-sided, flat-bottomed containers that should be left standing in an open area unaffected by buildings, trees, etc. For these receptacles, a direct measurement of water depth is satisfactory. If container has been emptied without measurement, it is desirable to have the observer refill the container and then measure the depth in the presence of the interviewer. This is also an effective check on the observation, if the measurement was made previous to the interview. The water depth is measured by inserting a thin ruler or steel tape vertically into the container. If

STORM PRECIPITATION SURVEY

Date 5/16/47

Storm Period April 27, 1947 Interviewer E. Dixson

General Location Yuma County, Colorado, 12 miles due west of Vernon

Township 1 S Range 46 W Section 28 1/4 Section NW Elevation approx 3800

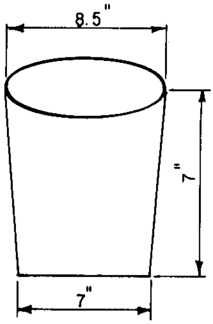
Observer Marvin Tuell Address Vernon, Colorado

PRECIPITATION

Began		Ended		Measured			True
Date	Time	Date	Time	Date	Time	Depth	Depth
4/27	3:30 PMST	4/27	DN — ST	4/28	6a MST	2.5"	1.8"
	— ST		— ST		— ST		
	— ST		— ST		— ST		
	— ST		— ST		— ST		
	— ST		— ST		— ST		

Describe receptacle and exposure; give details if: not level, sides not vertical (sketch), not empty before storm, not watertight, overflowed, poorly exposed; give distance from, and heights of trees and buildings. Quote observer on intense precipitation, beginnings, endings and amounts; form of precipitation, etc. Give further comment on reliability, ground condition, crops, etc.

Pail level, in yard, 50 feet south of tree and road. Burst of heavy rain with hail, 4:30 p.m. to 5:20 p.m.
Grass very short: 70% pasture, 15% wheat



$$V = \frac{A_B + 4A_M + A_T}{6} d \qquad A_B = \frac{\pi}{4} 7^2$$

$$V = \frac{\pi}{4} \frac{(49+211.4+56.9)2.5}{6} \qquad A_M = \frac{\pi}{4} 7.27^2$$

$$V = 33.1\pi \qquad A_T = \frac{\pi}{4} 7.54^2$$

$$A = \frac{\pi}{4} 8.5^2 = 18.1\pi$$

$$\text{True Depth} = \frac{V}{A} = \frac{33.1}{18.1} = 1.83 \text{ in.}$$

Figure 2-1.—Storm precipitation survey form. 103-D-1905.

container was not empty at the beginning of the storm, the beginning depth should be subtracted from the total depth at the end of the storm. If several observations were made during the storm, the rainfall between observations may be similarly determined. If receptacle is described as empty at beginning of storm, it should be indicated whether this is a matter of observation or a reasonable assumption. If receptacle overflowed, the time at spillover should be noted.

- For precipitation caught in irregular-shaped receptacles, the most direct method of determining the depth is to divide the volume of water (cubic inches) by the receptacle opening or catchment area (square inches) to obtain the depth (inches).

- For receptacles having a rolled edge, the catchment area is taken as extending to the ridge of the roll. In cases where volume or weight cannot be readily determined, use the prismoidal formula to find the measured volume of water in containers with sloping sides. For this method, it is necessary to know the horizontal area at the bottom, at half-depth of water, at surface of water, and at catchment level. The basic data usually required are height, top dimension, bottom dimension, and depth of water. The volume can then be computed:

$$V = \frac{A_b + 4\ A_m + A_t}{6}(d) \qquad (1)$$

where:
V = volume of water in cubic inches,
A_b = cross-sectional area at bottom in square inches,
A_m = cross-sectional area at half-depth in square inches,
A_t = cross-sectional area at water surface in square inches, and
d = depth of water in inches.

The volume V in equation (1) must be divided by the catchment area to get the true depth of precipitation.

- Presentations in the supplemental precipitation survey report should include a sketch of the receptacle showing dimensions, shape, direction, and amount of tilt; and location of receptacle with respect to obstacles and other potential shelters. An obstacle may exert effects on precipitation for a horizontal distance two or three times the height of the receptacle.

- Each team member should carry a steel tape or rule and appropriate graduates or measures. Liberal use of photographs cannot be overemphasized to add to the written descriptions. However, the interviewers should understand that photographs cannot replace the detailed sketches with dimensions of the receptacles.

2.4 Field Reconnaissance of Drainage Basins for Flood Hydrology Studies

As equally important as the basic hydrologic and meteorologic data in conducting a flood study is the accumulation of data regarding the physical features of the drainage basin being studied. Adequate accumulation of these data can only be accomplished by hydrologic engineers conducting a field reconnaissance of the basin. Studies conducted solely on the basis of office use of USGS quadrangle maps without physically visiting and inspecting the basin have led to many erroneous hydrologic analyses.

(a) General.—The field reconnaissance should be conducted prior to the initiation of any flood hydrology study. This includes any reconnaissance conducted at the appraisal level because these estimates are often used for later studies, and are not upgraded due to lack of time or funds. The purpose of the reconnaissance is to identify and document, in a trip report, pertinent physical features of the basin that may have an impact on the magnitude and timing of flood runoff, including existing water control facilities and existing and potential development.

(b) Participation in Field Reconnaissance.—In the interest of familiarizing all levels within the Bureau regarding a basin's hydrologic characteristics, representatives from the hydrology staffs of the project and regional offices and the Denver Office's Flood Section should participate in the field reconnaissance. At least one of the participants should be a senior level flood hydrologist with extensive experience in this specialty area. The advantage of this wide representation lies in the fact that based on direct, simultaneous, observations, all critical hydrologic parameters to be used in the flood analysis can be agreed upon in the field where the actual conditions are immediately apparent. When these parameter agreements are made at an early stage, the time to complete the study is shortened due to the elimination of uncertainties as to acceptable parameter selection that may arise in the course of conducting the flood study. Also, the time for review and approval of the study by the Denver Office is shortened because the necessity for technical revisions will essentially be eliminated.

(c) Field Observations.—The field reconnaissance team will observe and document the following four primary characteristics of the drainage basin:

(1) *Drainage network or "hydraulic system" of drainage basin.*—Particular emphasis should be placed on observing and documenting the hydraulic roughness characteristics of the drainage network of the basin. These characteristics have a direct impact on the drainage basin's hydraulic efficiency and, in turn, on the selection of unit hydrograph parameters discussed in chapter 4. The characteristics are most readily determined by visually inspecting representative reaches of the network and assigning average Manning's n values to each reach. It should be kept in mind that the n values assigned are to reflect water levels or stages that would be expected during extreme flood conditions. These observations should specifically consider overbank flow because these reaches are likely to be less hydraulically efficient than the main channel, meander

cut-off that shortens the length of travel of flood waters, scour potential, and the time of year the flood is likely to occur. The n values selected and the stream reaches should be called out and delineated on the maps used in the field reconnaissance. As will be discussed in chapter 4, these values will be averaged and will form the basis for selecting appropriate coefficients for the general unit hydrograph lag equation. An excellent guide for the selection of Manning's n values is the USGS Water Supply Paper 1849, *Roughness Characteristics of Natural Channels* [17]. This document provides Manning's n values, that have been determined from measurements during and after major flood events, for a variety of natural channel and overbank conditions. It also provides excellent colored photographs of the measuring sections and associated channel reaches. A complete description of the channels and overbank areas as related to hydraulic conditions should be included in the trip report. The description should include a discussion on the type of channel (swale, well incised, straight, meandering, etc.); character of overbank area (heavily wooded, grass covered, bare rock, etc.); and the material comprising the channel bed (boulders, bedrock, cobbles, native soil, etc.) and, if appropriate, the overbank areas. This information is useful for reference in supporting the results and conclusions that will have to be made at a later time when study participants may not be available. Photographs, preferably in color, should always be included as supplementary information in each reconnaissance report, and should be appropriately included and referenced in the narrative portion of that report. It is desirable that these photographs be taken from bridge crossings or at bends in the channel to show both channel and overbank areas.

The density of the well defined channels comprising the drainage network should be observed and described in the field reconnaissance report. The description should be tied directly to available USGS quadrangle maps used in the field work. This description will necessarily be somewhat subjective, but can be enhanced by the inclusion of representative aerial photographs, if available. The report should also include information relative to the extent of overland flow. This type of flow occurs in those portions of any basin where runoff must flow generally in "sheets" prior to reaching a point where the flow becomes concentrated in a channel or swale. Generally, the denser the drainage network, the more limited the distance overland flow must travel, which results in a more rapid runoff.

Agreement should be reached during the field reconnaissance as to the subbasin breakdown to be used in the study. In most cases, this breakdown is required for basins larger than about 500 square miles and for those basins where significant topographic variations are present. However, there are cases where this size of basin will be exceeded, such as when analyzing basins of very large areal extent like the Colorado River Basin. Subdivision is generally required to properly apply the unit hydrograph approach when converting rainfall or snowmelt into a runoff hydrograph. Topographic variations usually occur when both mountainous areas and flat valley or outwash plains are present in the basin and each has a significantly different response to a given amount of rainfall.

(2) *Soil and geologic conditions.*—Soil conditions should be observed and documented on a suitable map in terms of types of soil comprising the drainage basin and the areal extent of each type. The soils observed to be present in the drainage basin should be classified as to type using the four general SCS types discussed in chapter 4. Systematic observations and adequate documentation of these observations will provide the basis and support for selecting appropriate minimum or ultimate infiltration rates commonly used in the development of probable maximum and other flood hydrographs.

The general geologic setting, as it relates to runoff, should be described in the report. Because the reconnaissance team members will probably have only limited backgrounds in geology, the description of the geologic setting will have to be researched and extracted from the technical literature. Such literature is generally available from other related Bureau studies or USGS libraries. The geologic setting will, in many cases, have an impact on the selection of infiltration loss rates. In many areas of the United States underlain with limestone beds, depressions in the land surface have developed. These depressions, called "sink holes" or "playas," usually result in areas that will not contribute to runoff from a drainage basin. If such features are present in a drainage basin, they can have a significant effect on the flood runoff that can be expected because they will act as small reservoirs either detaining or storing runoff during flood events. Therefore, it is of prime importance that such areas be identified, delineated on a map, and assessed as to their floodwater storage capability. Also, such features should be fully described in quantitative terms in the field reconnaissance report. This description should be supplemented with color photographs as appropriate.

(3) *Vegetal cover.*—Another factor important to the satisfactory selection of estimated infiltration loss rates and unit hydrograph parameters is a general knowledge of the vegetal cover of the drainage basin. Therefore, during the field inspection, it is necessary to observe and document the types of vegetation present in the basin, particularly in the channels and overbank areas; and the areal extent and location of each type. Ground observations supplemented with aerial photographs have been found to be the most satisfactory way to accomplish this task. The results should be delineated on the USGS quadrangle map used in the field inspection, and should be discussed in the trip report, with the inclusion of color photographs.

(4) *Land use.*—Drainage basins tributary to Bureau dams are often natural or undeveloped basins. If this is the case and the basin is expected to remain undeveloped, it should be so stated in the field reconnaissance

report. However, there are an increasing number of situations where all or a portion of the basin will be used for agricultural purposes, including both crop production and livestock grazing; forestry, such as the tree harvesting that is prevalent in the Pacific Northwest region of the country; and urban development. In the case of agricultural and forestry land uses, the type, extent, and intensity of such uses should be determined during the field inspection and properly documented in the field report. Regarding the degree of urbanization, only development that presently exists can be observed and documented. Since dams will be in existence for an extended time, estimates of projected urbanization must be obtained along with an estimate of the storm drainage facilities that will be constructed in connection with the urbanization. Therefore, when conducting the field reconnaissance of a basin that includes or is near an expanding urban center, the local government entity responsible for land use planning and zoning should be contacted and a projected land use map secured for use in the flood study. Knowledge of projected urban land use is of considerable importance because the rainfall-runoff response of an urbanized drainage basin is very likely to be entirely different from that of the same basin in a nonurbanized condition. For example, in a relatively flat area in central Texas, the peak discharge from essentially the same rainfall from a particular basin was found to increase by a factor of almost 8 after being completely urbanized.

2.5 Examining Nearby Basins That Have Experienced Significant Recorded Floods

When the route of travel to or from a specific basin being inspected passes near a basin where a significant flood event has been recorded, time should always be allowed for a reconnaissance of that basin. Observations of the types previously discussed should be made and documented in the reconnaissance report. The documentation of these observations may serve as a basis for confirming hydrologic parameters used in the flood study for the drainage basin being analyzed, or for other ungauged basins within the hydrologically homogeneous region. Particular emphasis should be placed on conducting field reconnaissances of the basins described in chapter 4 to develop the general unit hydrograph lag time relationships for the six regional categories.

Chapter 3
HYDROMETEOROLOGY

3.1 General Considerations and Background

The "design storm" is the estimate of the rainfall amount and its distribution over a particular drainage basin, with respect to both time and area, that is used in the development of a PMF hydrograph or; in some instances, a flood hydrograph representing a specific frequency event. The purpose of this chapter is to provide the Bureau hydrologic engineer a basic familiarity with the atmospheric processes that produce extreme precipitation events, and the methods by which these processes are used in computing individual drainage PMS (Probable Maximum Storm) values and in developing regionalized criteria used to compute such values for particular drainage basins.

In 1981, the Bureau, NWS, and COE adopted a mutually acceptable, uniform definition of the widely used term PMP (Probable Maximum Precipitation) as "theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year." Complete technical application of this definition results primarily in a storm isohyetal pattern; isohyets are lines of equal precipitation that are analogous to contour lines on a topographic map. This pattern is overlaid and critically centered on a drawing of the drainage basin outline, and then the average precipitation is computed. The average basin precipitation is then distributed over time, which results in incremental basin average precipitation values that are used to generate the PMF hydrograph discussed in chapter 4.

The average basin precipitation is highly dependent on the total understanding of atmospheric processes as to the cause of severe event precipitation and the analysis of accumulated basic storm data. As hydrometeorologists expand their knowledge of severe storm meteorology, future revisions to present PMP estimates can be expected. However, at least for the conterminous United States, only minimum modification to current values of PMP is expected in the foreseeable future because knowledge of severe storm phenomena has reached a plateau.

Some discussion among meteorologists has centered on the effect of general climatic variations on estimates of PMP. Climatic trends are not incorporated into estimates of PMP for the following two reasons: (1) such trends progress rather slowly or fluctuate to an extent that their effect on PMP is small, especially in relationship to the other uncertainties associated with PMP development; and (2) effect of climatic variation on the production of extreme precipitation has not been fully agreed upon by members of the meteorological community.

The terms PMP and PMS have often been used interchangeably. For certain studies, these terms are used synonymously; e.g., those studies concerned with small area, short duration rainfalls. However, for the majority of studies, a precise distinction is realized. By definition, PMP refers to the "maximum precipitation amount for any area size and duration at a particular location during a certain time of the year." The PMS results from adjustments applied to the PMP that provide a total design storm event, realistically patterned (spatially-temporally) after actual storms of record, and that has been adjusted to a single PMP area size and duration deemed critical for design. An example of such adjustments is the use of within/without storm DAD (depth-area-duration) relationships as described in the PMP applications report HMR NO. 52 [18][1]. It should be noted that these adjustments will tend to result in a lower average basin rainfall value. The adjustments are negligible for very small basins and significant for very large basins.

Values of PMP are called "estimates" because there is no direct means of computing and evaluating the accuracy of the results. Judgments based on meteorological principles and storm data are the most important factors in assessing the level of PMP. The lower limit of PMP is set by observed storms of record. In evaluating the appropriate level of PMP, it is necessary to account for such factors as the length of record available, number of severe storms that have occurred, degree of envelopment of PMP over observed data, transposition limits to severe storms of record, maximizing steps incorporated, validity of models used for assessing PMP, and care that excessive compounding of the meteorological factors associated with rare events has been avoided. The evaluation is often complicated because of the uniqueness of the problem presented. No two basins provide exactly the same problems, and the available data base of severe storms is hardly sufficient for most regions. Because of these variable conditions, a "cookbook" approach to PMP determination and its evaluation is not available. Hydrometeorologists charged with the task of determining PMP must appropriately weigh what has been previously derived for past specific studies with what their combined meteorological judgment provides. As our knowledge of the severe storm process expands, it is expected that present-day estimates of PMP will be refined. The Bureau's current approach in determining PMP is the adoption of material obtained in various hydrometeorological reports issued by the Hydrometeorological Branch of the NWS. Although modifications to these reports are sometimes necessary, it has been determined by Bureau hydrometeorologists that these reports provide the most accurate and consistent method of PMP derivation. For these reasons, PMP derived using the HMR series is the method used by the Bureau where applicable.

[1]Numbers in brackets refer to entries in the Bibliography.

3.2 Atmospheric Processes

To more fully understand the derivation of PMP, some knowledge of the precipitation process should be useful. Basically, four conditions are necessary to form intense precipitation: (1) abundant atmospheric moisture, (2) a lifting mechanism, (3) condensation, and (4) water droplet-ice crystal growth.

(a) Atmospheric Moisture.—The term "atmospheric moisture" refers to water vapor only and not to water in either its liquid or solid state. Water vapor in the atmosphere is commonly measured in the United States in terms of inches of precipitable water. Precipitable water is defined as "the height of condensed total atmospheric water vapor contained in a vertical column of air of unit cross section located between any two levels of the atmosphere" [19]. It should be noted that, in severe storms, precipitation often exceeds the measured precipitable water depth of the total overlying atmosphere. This situation is explained by the fact that, during the storm event, additional moisture is constantly being fed into the area due to the nature of the storm circulation. Measurements of atmospheric moisture through depth are typically made by radiosonde and/or weather satellite related observations.

Water vapor is always present in the atmosphere, with the maximum amount limited by air temperature. Air becomes saturated when it contains the maximum amount of water vapor at a particular temperature. Saturation is achieved by either the addition of moisture or the cooling of air to a lower temperature, where the maximum amount of water vapor is held and condensation begins to occur. Although both processes work simultaneously during the formation of precipitation, it is the latter that has the greatest effect in producing intense precipitation. The temperature to which a given parcel of air must be cooled at constant pressure and constant water-vapor content for saturation to occur is called the "dewpoint". Air at a temperature near the dewpoint temperature is likely to produce condensation particles.

(b) Lifting Mechanism.—Precipitation is basically caused by the cooling of moisture-laden air to a temperature at which the atmosphere cannot retain its moisture charge (saturation). To produce intense precipitation, nature achieves the required rapid cooling by air expanding as it rises or is lifted into the atmosphere. The total process that causes this upward forcing of air and resultant cooling is termed the "storm lifting mechanism."

Four separate storm lifting mechanisms can be distinguished that provide the necessary lift for large quantities of moist air [20]. During actual storm occurrences, from one to all four mechanisms may interact to produce resultant saturated air and eventual precipitation. These four lifting processes are briefly described as follows:

(1) *Atmospheric convergence.*—Lifting by this mechanism results from the interaction of atmospheric pressure and circulation to concentrate moist air. The convergence that takes place near the Earth's surface forces the air upward, where expansion and cooling take place. Precipitation formed mainly through this process can be intense over large areas and for long durations. Such precipitation is often related with extratropical (storms that originate outside the region of tropical easterlies) and tropical storm types that are readily interpreted on a synoptic (large area) scale of analysis.

(2) *Orographic lifting.*—When moist air flows up a range of hills or mountains, orographically induced precipitation may occur. The rate and quantity of air cooled is dependent on the steepness and orientation of the barrier encountered relative to the direction and magnitude of the inflowing moist air mass.

(3)*Fronts.*—A front is described meteorologically as the interface between two air masses of different density. An air mass is a large body of air that is almost homogeneous in horizontal direction, vertical temperature and moisture variation are the same over its horizontal extent [19]. Air masses become established while situated over a particular region of the Earth's surface for prolonged periods. The surface of contact between air masses is an inclined surface with the lighter warmer air above and heavier colder air below. When a colder air mass is displacing warmer air; i.e., when front is moving in direction from cold air to warm, a cold front results. A warm frontal surface is one in which warm air is displacing cold, and this surface has a gentle slope as the less dense warm air smoothly overruns the retreating cold air mass. Because of earth surface frictional forces, a cold front is steeper in the lower atmospheric levels than that established by a warm front. Because of the shallow surface presented, neither a cold nor warm front can produce significant precipitation.

Frequently, a frontal wave is developed along a front when the movement of a portion of the front is retarded and a counterclockwise wind circulation (northern hemisphere) develops. This occurs most frequently when a low pressure trough in the upper atmosphere, commonly termed a "short wave", overtakes the surface frontal system. Under favorable conditions, usually in concert with one or more of the other storm lifting mechanisms, a wave may develop rapidly and cause significant precipitation. Storms developed in this manner occur mainly in temperate climates and are known as "extratropical cyclones."

A series of waves may be generated when a front does not indicate significant motion toward either the cold- or warm-air mass. Such frontal positioning is known as a "stationary or quasi-stationary front". A series of small waves, generated under these conditions, may successively cause precipitation over a given area and may even produce a major storm even though none of the waves could be regarded individually as a major storm system.

(4) *Convection / Instability.*—An extremely important concept in the production of extreme precipitation is that of convection/instability. Meteorologists predominantly refer to convection as "mixing of atmospheric properties in the vertical direction." A further distinction is made between forced and free convection. Forced convection can be demonstrated by the previously mentioned three storm lifting processes of atmospheric convergence, orographic, and fronts. Free convection arises from atmospheric instability, which can be defined as "a condition that if a parcel of air is less dense than its surrounding environment, it will become buoyant;" i.e., the parcel will continue to rise and therefore eventually cool to its saturation temperature. A parcel of air that has reached saturation will continue to cool with increasing elevation at a rate less than unsaturated air. This results from the release of latent heat due to condensation. Therefore, a parcel of saturated air in an unsaturated environment will remain warmer than its surrounding air with increasing height and continue to rise, maintaining saturation through substantial depth often producing intense precipitation over small areas for short durations.

Atmospheric instability is present to some degree during any severe storm event. Combined with the previously mentioned storm lifting mechanisms, derived precipitation can be intense over large areas for long durations. Other storm lifting mechanisms often provide the initial impetus (lift) to place moisture-laden air into a unstable environment, which causes explosive cloud development and major precipitation.

An excellent example of convective activity is the development of local storm (thunderstorm) precipitation. A local storm is defined as "an isolated precipitation event restricted in both spatial and temporal distributions." Local storms are typically generated by the heating of air near the Earth's surface. If atmospheric instability is present through significant depth, rapid vertical cloud growth takes place, resulting in showers and thunderstorms. Local storms involving widespread activity of this type are of maximum frequency and magnitude during the summer when maritime tropical air masses occur frequently and when the heating of air near the ground is sufficient to initiate convective activity.

(c) Condensation.—When water vapor is transformed into a liquid or directly into a solid (sublimation), the process is called "condensation." The formation of dew, fog, and clouds are typical visual results, and achieving saturated air does not necessarily mean condensation will occur. Additional latent heat of condensation must be removed from a parcel of saturated air, and condensation and/or freezing nuclei must be available in sufficient quantity before droplets of water or ice crystals are formed. Condensation nuclei are hygroscopic particles that act as a catalyst upon which water vapor condenses. Such airborne particles are dust, sea salt from evaporated sea spray, and combustion nuclei.

(d) Water Droplet-Ice Crystal Growth.—Condensation of water vapor to liquid droplets or ice crystals results in particles of such size that only the slightest updraft is required to suspend them in the atmosphere. To overcome the substantial updrafts that are associated with extreme precipitation events, another process must occur to further the growth of the initially formed droplets/crystals. Theories explaining such growth are found in most meteorological texts concerning cloud physics. The two most advanced explanations are the Bergeron-Findeisen theory (ice-crystal) for cold cloud (temperature less than 0 °C) and the collision-coalescence theory for warm cloud (temperature greater than 0 °C) particle growth.

In the Bergeron-Findeisen theory, both ice crystals and liquid water droplets exist together in a supercooled cloud. At some subfreezing temperature, the equilibrium vapor pressure of water vapor to ice is less than that to liquid water, and ice crystal growth will then occur at the expense of the water droplets. Such growth continues until the ice crystals become large enough to fall. Further growth occurs as the ice crystals collide with other crystals (aggregation) and with supercooled cloud droplets (riming).

The collision-coalescence theory is the merging of two or more water droplets into a single larger droplet or raindrop. Initial contact occurs from impact; however, whether the coalescence activity is completed and a single large drop is formed depends on the relative velocity of impact, size and concentration of drops, and the electric charge of individual drops and surrounding field. Once sufficient raindrop mass is obtained so that gravity overcomes the storm updrafts, precipitation finally occurs.

It is entirely possible at mid and high latitudes for one or both of the theories to form precipitation. At low latitudes (tropics) where rainfall frequently occurs from warm clouds, only the collision-coalescence theory offers a reasonable explanation for cloud droplet growth. However, in deep storm clouds, the ice crystal processes are valid at low latitudes.

In summary, the quantity of precipitation formed is dependent upon the following three primary factors: (1) amount and rate of inflowing moist air into the storm area, (2) rate of cooling (updraft) produced by lifting mechanisms, and (3) the rate of growth of cloud droplets/ice crystals to form precipitation-size hydrometeors.

3.3 Derivation of PMS

In the development of the PMS, the most critical element is the calculation of the magnitude, duration, and areal extent of the PMP. In the deterministic approach to evaluating PMP, two methods are generally applicable. The first method uses the "storm maximization" approach, and the second uses the "storm model" approach. Occasionally, aspects from both approaches are combined to produce appropriate results.

(a) Storm Maximization Approach.—Section 3.2 indicated that the amount of precipitation formed is basically controlled by the rate at which available moist air can be processed by a storm. The storm maximization approach uses the information described in section 3.2, and is based on the following two assumptions: (1) precipitation can be determined from the product of available moisture and storm mechanism, and (2) that the record of severe storms is sufficiently large that an optimum "storm mechanism" has been realized. This storm mechanism includes all the conditions described in section 3.2 for the formation of precipitation except for atmospheric moisture. The reasoning supporting these assumptions is twofold. First, studies have shown that the amount of precipitation obtained is directly related to the quantity of atmospheric moisture available. Second, due to the lack of sufficient measurements during actual storm events to quantify each of the conditions causing precipitation, excluding moisture content, the solution is to use the record of extreme precipitation amounts as an indirect measure of the maximum potential of these conditions. By adjusting observed severe precipitation amounts to maximum moisture conditions, the greatest precipitation potential for that particular storm event is assumed to be determined. The lack of a sufficient storm data base at individual locations is compensated for by the introduction of storm transposition and envelopment to achieve the level of PMP. Brief descriptions of the data base and operations of storm moisture maximization, transposition, and envelopment performed upon this data base are given in the following paragraphs.

(1) *Data base.*—The basic data are the records of the largest known observed areal-duration precipitation amounts. These data are developed by the standardized DAD procedures described in [21]. The DAD values represent the average depth of precipitation that has occurred over an area of given size within a specified time interval. For any given storm, these values represent the highest average depth for selected area sizes and durations.

For the United States, DAD type data has been assembled for more than 500 such storms, and has been accumulated in an open-ended publication [23]. A set of similar but unofficial data for about 300 additional storms analyzed by the Flood Section of the Bureau, Hydrometeorological Branch of NWS, or found in the literature [24], supplements the basic data in reference [23]. For drainages situated near the Canadian border and the United States, the Canadian Atmospheric Environment Service has prepared similar analyses of 400 storms in Canada [25].

(2) *Moisture maximization.*—Moisture maximization is a process whereby observed storm precipitation is increased to a value consistent with the maximum moisture in the atmosphere for the storm location and time of year. Moisture maximized precipitation has the same ratio to the observed storm precipitation as the maximum moisture charge has to the moisture charge of the observed storm. It has been mathematically shown, on a diverse set of storm models, that the rainfall was closely proportional to the moisture charge available for each model [20].

The moisture charge in an air mass is expressed in terms of precipitable water. In the determination of a storm moisture maximization factor, the surface dewpoint temperature, in conjunction with an assumed saturated atmosphere above surface level, is used as an index of available precipitable water. Tests performed with data from severe storms of record indicate adequate support for this assumption [20]. Surface dewpoints, instead of data collected by radiosonde, are used as a measure of available moisture because they are the only measure of moisture potential for early severe storms of record (prior to 1940), plus the density of stations taking such observations is large enough to sense narrow tongues of inflowing moisture often critical to small area severe storm development [26].

In the storm maximization process, dewpoints are chosen as the highest value persisting for 12 hours. It is believed that this time period is more representative of inflow necessary to establish severe precipitation, as well as reducing the error of instantaneous observations.

Dewpoints are comparable only if they represent conditions under the same atmospheric pressure; i.e., to basically normalize dewpoints for differences in station elevations. The standard is to adjust all surface dewpoints to a pressure of 1,000 mbar (millibars), which is approximately sea level pressure. The adjusted sea level dewpoint is the temperature that a parcel of air would achieve if cooled to the dewpoint at observed pressure, and then compressed adiabatically to 1,000 mbar, with moisture supplied to keep the air saturated during compression. Such a process is called pseudoadiabatic (saturation-adiabatic), and is represented by the solid sloping lines on figure 3-1.

Figure 3-1 is used in converting an observed dewpoint to its sea level equivalent. Initially, a point corresponding to the elevation (ordinate) and observed dewpoint (abscissa) is plotted; a curve is then traced through that point parallel to the adjacent pseudoadiabats. The sea level dewpoint is read at the intersection of the curve and the zero-elevation line (sea level). For example, a dewpoint of 58 °F at 5,000 feet is equivalent to a dewpoint of 70 °F at sea level.

The actual moisture maximization factor formed is the ratio of maximum precipitable water to water available during the storm, based on associated dewpoint temperatures and a saturated atmosphere. For a saturated atmosphere, tables of precipitable water have been prepared

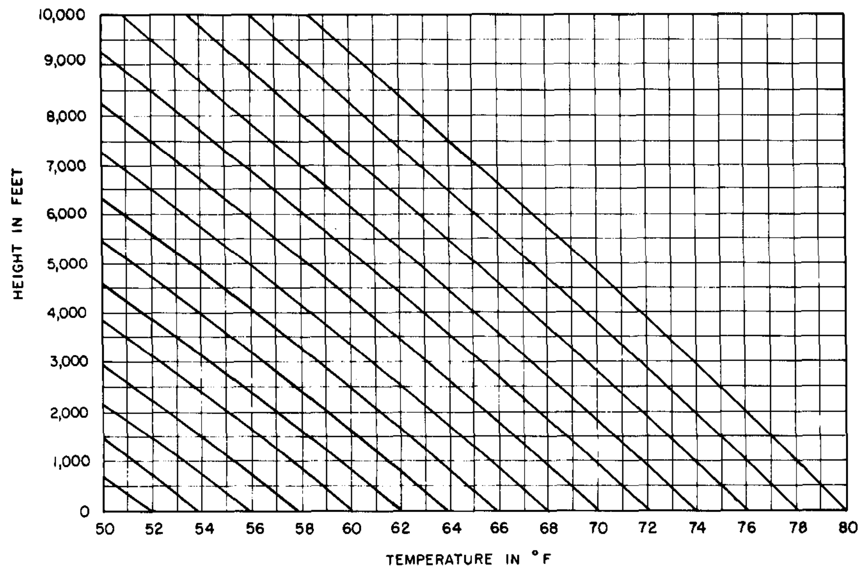


Figure 3-1.—Pseudo-adiabatic chart for dewpoint-elevation adjustment. 103-D-1906.

providing such measurements based on 1,000-mbar dewpoints [27]. Figures 3-2 and 3-3 show the results of these tables in nomogram form [28]. As an example using these figures, assume it is desired to determine the total precipitable water in a column of saturated air from sea level pressure (0 feet) to the 200-mbar level (about 40,000 feet) having a sea level dewpoint of 70°F. Locate the point on figure 3-3 at the intersection of the 200-mbar surface (ordinate) and the 70°F temperature (abscissa, scale along top of page). Since saturation is assumed, air temperature is equivalent to dewpoint (saturation) temperature. From this intersection, proceed vertically downward to read precipitable water (abscissa, scale along bottom of page) value of 2.27 inches.

Determination of two dewpoints is necessary for moisture maximization calculations. The first is the "representative storm dewpoint", which is a measure of the moisture inflow in an actual storm. The second is the maximum dewpoint for the same location and time of year when the storm occurred.

Representative storm dewpoints are selected in the moist air flowing into the precipitation area. It is necessary to record both distance and direction of the selected dewpoint location in relation to the precipitation center. Details of guidelines used in the selection of representative storm dewpoints are available in several publications [29,30,31]. For a particular storm, standard practice is to average dewpoints from several stations, over the same time period, each reduced to 1,000 mbar and persisting

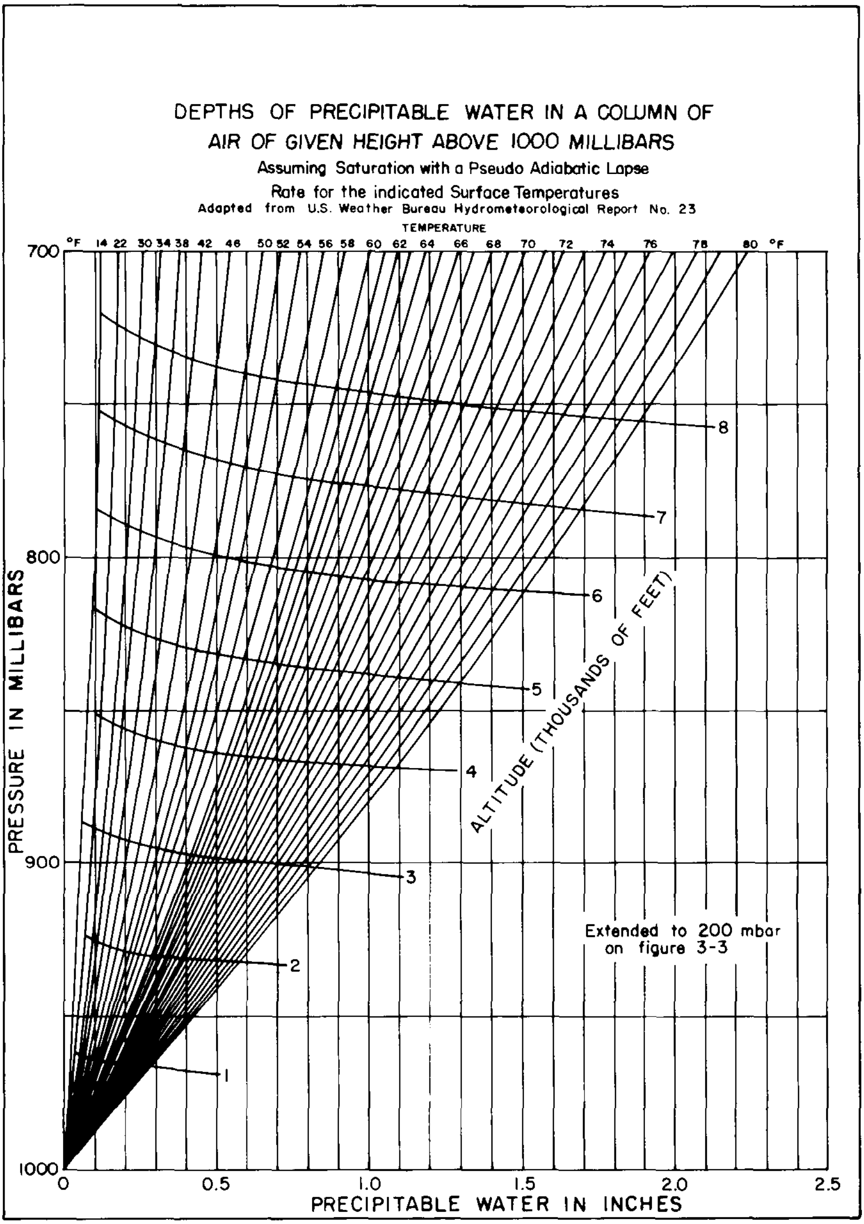


Figure 3-2.—Diagram for precipitable water determination from 1,000 to 700 millibars. From [28]. 103-D-1907.

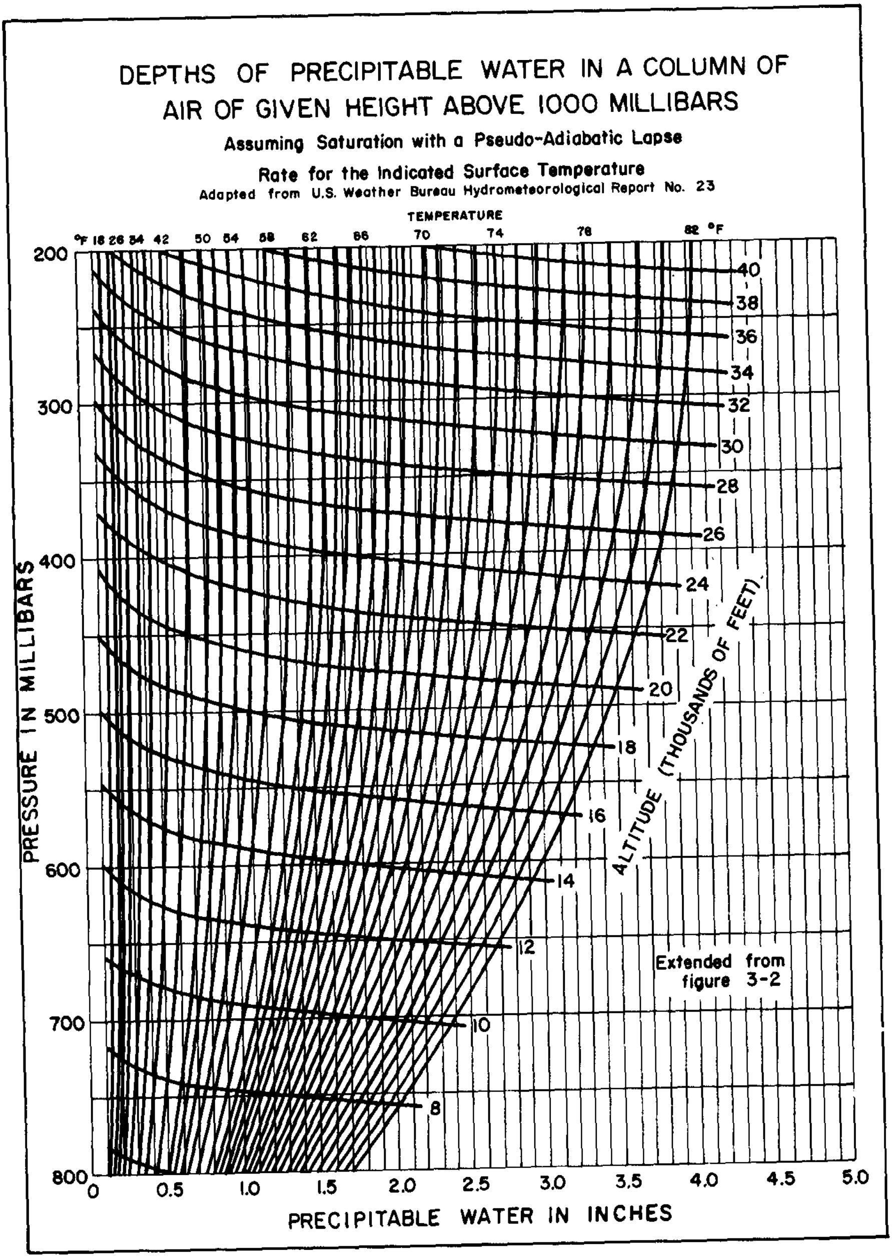


Figure 3-3.—Diagram for precipitable water determination from 800 to 200 millibars. From [28]. 103-D-1908.

for 12 hours as representative of individual storm moisture. A representative storm dewpoint is chosen upwind of the precipitation area. A distance is selected that allows the greatest moisture content available to be processed during the storm; therefore, the distance and direction of the selected representative dewpoint must be compatible with observed lower level winds and time of dewpoint selection.

Maximum dewpoints typically express the highest dewpoint observed for a particular location and time of year that occurs in the vicinity of associated rainfall potential. The exact location of the maximum dewpoint used in the moisture maximization process is the same location used for the representative storm dewpoint. Charts of the largest persisting 12-hour, 1,000-mbar observed dewpoints that are regionally and seasonally smoothed to provide a maximum analysis are found in several publications [32,33,34], dependent on the region of interest.

The selection of the maximum dewpoint is normally taken 15 days into the warm season (higher dewpoint) from the date of the representative storm dewpoint occurrence. This method accounts for the assumption that a critical storm could reasonably have occurred 15 days before or after its storm date without appreciably affecting the storm mechanism. Additional increases of up to 10 percent in adjusted storm precipitation are often realized from application of this procedure.

If a mountain barrier interrupts air flow from the moisture source to the precipitation region or the precipitation occurs at an elevation different from sea level, further moisture adjustments are applied. Under these conditions, a reduction to the available precipitable water above sea level due to the barrier or elevation is necessary. To determine the moisture charge available for a drainage basin at a given dewpoint under such circumstances, subtract the precipitable water in the column of air extending from sea level to the height of the barrier or precipitation elevation, whichever is greater, from the precipitable water in the total column for the same dewpoint from sea level to the 200-mbar level. This procedure is applied to both representative storm and maximum dewpoint related calculations of available precipitable water before determining the storm maximization factor. The basic storm maximization process can be expressed mathematically as:

$$P_a = \frac{P_o \, W_p \text{ (max.)}}{W_p \text{ (storm)}} \tag{2}$$

where:

P_a = adjusted moisture maximized precipitation,
P_o = observed precipitation, and
W_p = precipitable water (max. to maximum W_p and storm refers to representative storm W_p.

(3) *Storm transposition.*—Transposition involves relocating individual storm precipitation within a region considered homogeneous relative to topographic and meteorologic characteristics deemed significant to that storm. As each drainage basin of interest has not likely experienced the number of severe storms necessary for PMP development, transposition becomes an important tool for providing additional data at a particular site. In deriving the PMP, two types of storm transposition are used: (1) "explicit" transposition, which is usually defined as "the region where a particular storm may be transposed without experiencing significant change to its storm mechanism." An exception to this definition would be the distance from the coast adjustment for tropical storms. Adjustments based on moisture availability are normally the only modifications permitted within the explicit transposition region, and (2) "implicit" transposition. Knowing that nature would not typically permit atmospheric discontinuities to occur at limiting boundaries to explicit storm transposition, implicit transposition (or regional smoothing) is applied. Implicit transposition extends the explicit limits beyond what would normally be determined, and is applied only if meteorological or topographical explanations of potential discontinuities cannot be postulated. Details of storm transposition procedures are available in several publications [29,31,35,36,37], so only general comments concerning these procedures will be made in this manual.

Storm transposition begins by clearly identifying the location of the precipitation, and then determining what atmospheric processes were at work to produce the event. A complete meteorological and topographical analysis is made by the hydrometeorologist, which leads to a classification of the particular storm type. Meteorological records are then researched and surrounding terrain features examined to identify similar regions where the storm could reasonably be transposed. Limits of transposition are then established. The final step in this procedure is the application of adjustments within the explicit transposition limits for the specific relocation of the storm.

The typical storm transposition adjustment is determined by the ratio of maximum precipitable water, see section 3.3 (a)(2), for the transposed location to the maximum available for the storm in place. The maximum transposed dewpoint is selected using the same distance and direction from the transposed storm location as was determined for the selection of the representative storm dewpoint. The moisture maximization and transposition adjustments can be expressed mathematically, and simplified to produce the moisture maximized-transposed precipitation as follows:

$$\begin{array}{c}\text{Adjusted moisture}\\ \text{maximized and}\\ \text{transposed}\\ \text{precipitation}\end{array} = \left(\begin{array}{c}\text{Observed}\\ \text{precipitation}\end{array}\right) \text{ times } \left(\begin{array}{c}\text{Moisture-}\\ \text{maximization}\\ \text{factor}\end{array}\right) \text{ times } \left(\begin{array}{c}\text{Transposition}\\ \text{factor}\end{array}\right)$$

$$P_{at} = (P_o)\left(\frac{W_p \text{ (max. in place)}}{W_p \text{ (storm)}}\right)\left(\frac{W_p \text{ (max. transposed)}}{W_p \text{ (max. in place)}}\right)$$

$$= (P_o)\left(\frac{W_p \text{ (max. transposed)}}{W_p \text{ (storm)}}\right) \quad (3)$$

where:
P_{at} = adjusted moisture maximized-transposed precipitation,
P_o = observed precipitation,
W_p = precipitable water (max. refers to maximum W_p in place or transposed location, and storm refers to representative storm W_p).

The adjusted precipitation, as expressed in equation (3), necessarily accounts for differences in horizontal or vertical (barrier) effects due to moisture maximization and transposition of observed precipitation to other locations. Horizontal transposition closer to the moisture source causes related increases to the adjusted precipitation, and horizontal transposition away from the moisture source causes related decreases to the adjusted precipitation. Increases in elevation or barrier considerations result in associated decreases to adjusted precipitation, and decreases in elevation or barrier considerations result in associated increases to adjusted precipitation. For additional transposition adjustments that may apply for special circumstances dependent on storm type, location, or available data, see references [29,31,35,38].

(4) *Envelopment.*—Envelopment involves the selection of the likely greatest value from a set of data. This step becomes a requirement due to the lack of a uniform storm data base for every duration, area, and location of interest. Envelopment accounts for the random occurrence and variation of individual severe storm precipitation. Figures 3-4 and 3-5 are examples of depth-duration and depth-area smoothing, respectively, of individual moisture-maximized and transposed-adjusted precipitation. Information taken from these figures are eventually combined in a complete DAD analysis, shown on figure 3-6. In line with the definition of envelopment, it should be noted from figure 3-6 that the enveloping lines do not necessarily pass through every moisture-maximized and transposed-precipitation amount. Also, note that the same storm does not control the enveloping curves for all the area sizes and durations indicated.

In the development of regionalized studies, section 3.3 (c)(2), additional regional smoothing of a number of individual analysis, similar to that shown on figure 3-6, are performed for the entire study area. For individual drainage analysis, section 3.3 (c)(1), indirect regional smoothing methods are often used to ensure that the appropriate level of PMP has been achieved and that consistency has been maintained with adjacent study basins. Inconsistencies in PMP estimates within or between regions need to be meteorologically or topographically justified, or such anomalies smoothed.

(5) *Summary.*—The three processes of storm moisture maximization, transposition, and envelopment performed on observed precipitation are the basis for developing the PMP using the "storm maximization

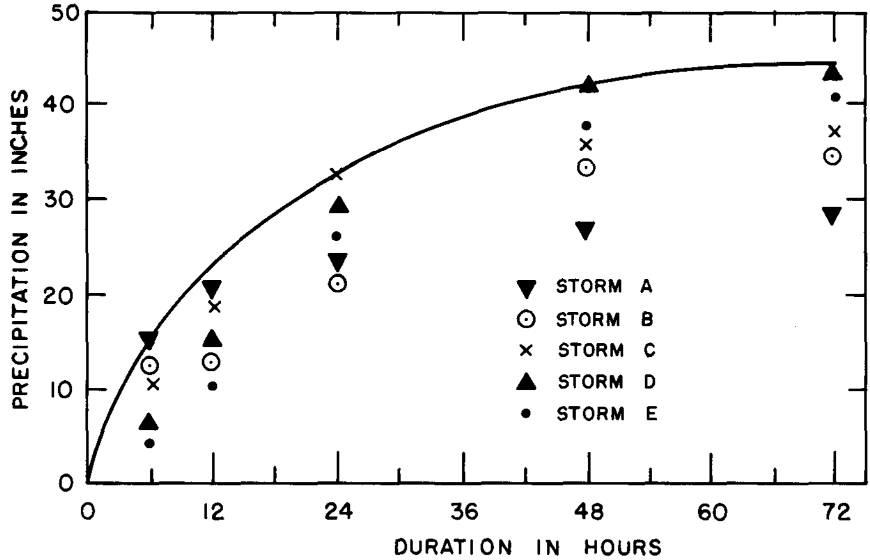


Figure 3-4.—Depth-duration envelope of transposed maximized storm values for 1,000 square miles. 103-D-1909.

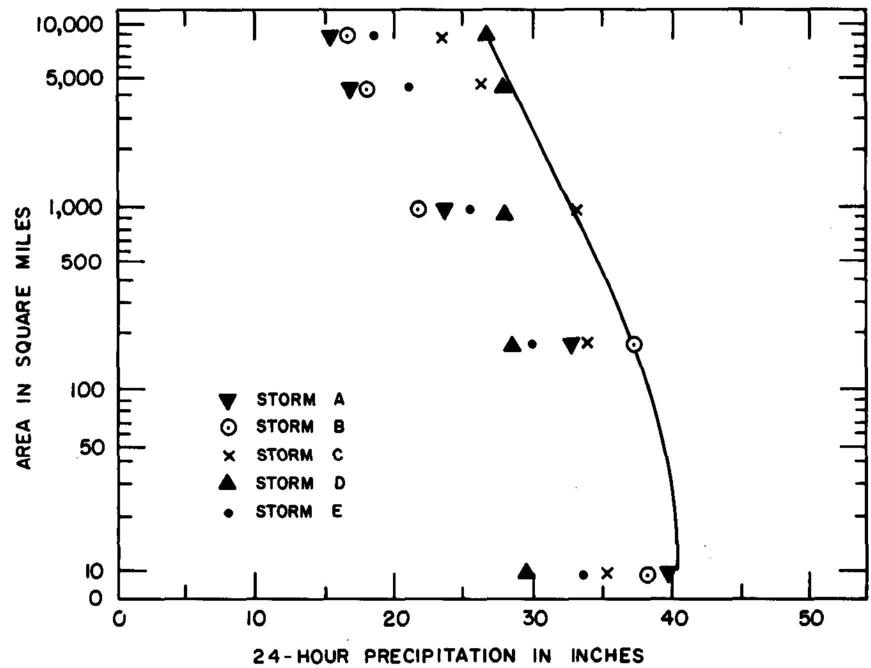


Figure 3-5.—Depth-area envelope of transposed maximized 24-hour precipitation. 103-D-1910.

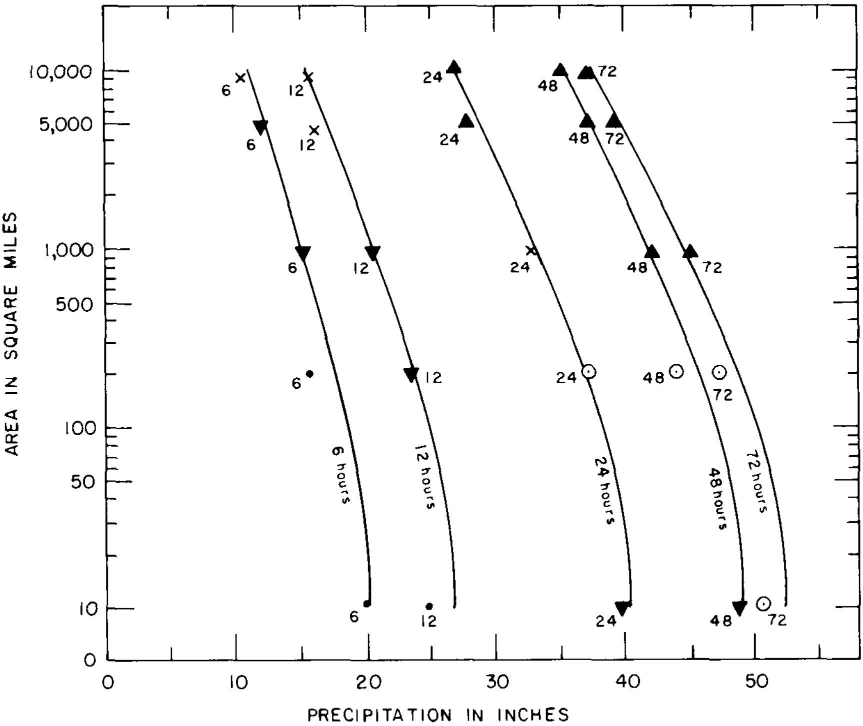


Figure 3-6.—Depth-area-duration envelope. 103-D-1911.

technique." Elements of these processes are also incorporated in the "storm model" approach, section 3.3 (b). Modifications to these processes are occasionally required, dependent on the study location and data available. Individual reports previously referenced should be consulted for details.

(b) Model Technique.—A second approach to a deterministic PMP evaluation is accomplished using storm models, where a model describing the various atmospheric processes of section 3.2 is hypothesized. Calibration of the model is obtained by varying, mathematically, the processes describing the storm mechanism and moisture availability until satisfactory comparisons are achieved with actual severe precipitation events of record. Having replicated precipitation from observed storms, the various parameters attributed to the storm mechanism and moisture inflow are then maximized in an attempt to realize the full PMP potential.

To date, satisfactory models describing PMP potential have not been fully realized, and only limited success using an orographic model has been attained [39,40]. Various difficulties encountered using models are discussed in several publications [29,38,39,40]. Limited use of an orographic model is described in section 3.3(c)(2).

It is generally accepted, among practicing hydrometeorologists concerned with severe storm precipitation events, that use of such models is the preferred method. It is expected that models will be developed that will eventually simulate the severe storms of record and be adapted to provide more reliable estimates of PMP.

*(c) **Individual Drainage Estimates and Regionalized Studies.***—The approaches to PMS development of storm maximization or modeling techniques can be applied to either an individual basin or to a large region that contains a multitude of drainages of varying size and shape. For the individual basin, the "individual drainage estimate" and the results are to be exclusively applied to the single drainage under study. For a large region, the term "regionalized" or "generalized" approach is used for the PMS evaluation. For the regionalized approach, an area of similar topographical and meteorological features is defined and the procedures of storm maximization and/or model technique are applied to portray the PMS in generic form for the entire region. The final result is obtained using appropriate figures, tables, and equations for which values of the PMS are obtained for any drainage located within the study area and within the limits (durational-areal) of the regionalized report. With few exceptions, the regionalized approach as set forth in the HMR report series is to be used in determining PMP and PMS values for PMF development. Usually, exceptions arise when the drainage basin being studied is larger than that for which criteria are presented in the report series.

Regionalized PMS criteria [18,20,29,31,35,39,40,41] are used because they possess several distinct advantages such as: (1) greatest use of available data can be incorporated, (2) storm maximization or model techniques provide a greater degree of reliability to the PMS if analyzed on a regional basis, (3) consistency among individual basin estimates is obtained, (4) individual estimates of PMS can be readily obtained from completed regional studies by hydrologic engineers, and (5) regionalization serves as a base of severe storm information and criteria to further develop individual drainage study requirements for specific locations when additional information becomes available.

The primary disadvantages of regionalized studies are: (1) time required to complete and document studies often take several years, (2) extensive manpower requirements that include several hydrometeorologists with a specialty in PMS criteria development, and (3) the scale of analysis is such that minor refinements are not incorporated because of the smoothing involved.

The Bureau's development of the PMS, unless obtained from regionalized reports [18,31,35,38,39,40,42,43,44,45], is always conducted by a professional hydrometeorologist in the Flood Section of the Bureau's Denver Office; or through consulting meteorologists in conjunction with hydrometeorologists on the Bureau's staff. The PMS estimates developed at various Bureau regional offices using approved regionalized criteria are reviewed by personnel in the Flood Section for compliance with regionalized criteria and recent agreements, revisions, or refinements.

(1) *Individual drainage estimates for PMS.*—To date, all individual drainage estimates for PMS development computed by the Bureau have been completed using the storm maximization technique by professional meteorologists. The procedure basically involves the considerations described herein, and is presented as background information for the flood hydrologist.

Use of the storm maximization technique for an individual drainage is based on the assumption that adequate data are available describing severe historic storms that have occurred in or near the basin of interest. Of these storms of record, a sufficient number must be considered transposable to the study basin with only limited modifications imposed, as described in section 3.3(a). Storms located within a homogeneous meteorological-topographical region, as the study basin, may be assumed to have the potential to occur over the drainage area. It is also assumed that several of these severe storms occurred having the most efficient storm mechanism, and could be brought to their full precipitation potential through application of the moisture maximization procedure.

Using the previous assumptions and the storm moisture-maximization and transposition techniques of sections 3.3(a)(2) and 3.3(a)(3), each storm's isohyetal pattern is transposed and fitted over the basin in the most critical position, considerations to pattern orientation may apply. In nonorographic regions, individual storm precipitation patterns are directly shifted from their inplace location, and judiciously reorientated over the subject basin. For orographic regions, the storm isopercental analysis technique, often used to develop inplace storm isohyetal patterns in data deficient orographic regions, is typically used in relocating the precipitation patterns to account for differences in complex terrain features between the inplace and transposed storm locations. Basically, the isopercental technique adjusts inplace storm pattern precipitation by the ratio of the analysis of precipitation given for some specific return period and duration at the transposed site to that evaluated at the storm inplace location. Occasionally, maps of mean annual precipitation are helpful in this adjustment. The technique works best for large area transpositions of highly orographically controlled storms where precipitation amounts are influenced by the same parameters that caused the variation in precipitation described by the selected base map at both the inplace and transposed locations. Care must be directed in the selection of appropriate geographical limits of transposability.

Basin average precipitation is determined by calculating the volume of precipitation that would have fallen over the drainage area after the storm was transposed. Timing of the precipitation in the basin follows the temporal distribution associated with the storm occurrence inplace, as developed in reports [21,46] concerning storm DAD analysis. Average basin precipitation is maximized using the ratio of maximum precipitable water at the transposed location to that available (measured) during the actual storm at its inplace location with appropriate considerations given to intervening barriers, elevation changes, and distance from moisture source.

Adjusted average basin precipitation, maximized for moisture and transposition, for each storm transposed to the study basin is plotted and analyzed in a smooth depth-duration plot similar to the one shown on figure 3-4. The incremental precipitation values from the smooth enveloping curve of individual storm adjusted data represents the maximum precipitation for the basin.

Adopted temporal distributions of incremental average basin precipitation, read from the smooth depth-duration curve, are rearranged after a selected storm of record. If spatial distribution is required, it may be accomplished using one or a composite of the isohyetal patterns that have occurred in or been transposed into the drainage. Typically, the temporal and spatial distributions are obtained from one or more of the storms controlling the depth-duration enveloping curve (fig. 3-4) for the study basin.

The PMS estimates derived from individual drainage analysis should be compared for consistency with estimates for similar drainages in the same homogeneous region. When appreciable differences are identified, they should be studied carefully, and justified or modified. For those results considered suspect, the various steps in deriving the PMS for the basin should be carefully reexamined. Consistency is difficult to maintain when PMS estimates are completed using the individual drainage storm maximization approach when applied by different individuals at various times. More consistent and reliable results are achieved using the regionalized criteria described in the following subsection.

(2) *Regionalized PMS.*—Regionalized studies leading to the eventual development of the PMS are currently available in the various HMR and Technical Paper series of publications issued by the NWS, see the list of references at the back of this manual. Methodologies and criteria obtained from these publications have been examined by Bureau hydrometeorologists and are considered to provide the best estimate of PMP potential within the limits of each report. Since 1980, Bureau personnel have participated with NWS hydrometeorologists in the development, review, and revision of the criteria available in several of these publications. The PMS derived from these reports and used in the derivation of the PMF is reviewed by Bureau personnel at the time the flood study is reviewed. Regionalized PMS reports available for the entire conterminous United States are regionally depicted on figure 3-7. Each report

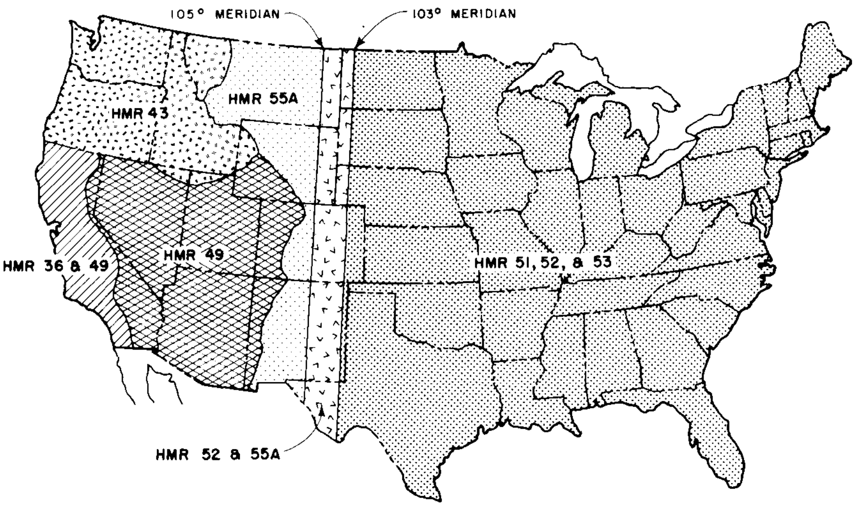


Figure 3-7.—Hydrometeorological report series coverage of conterminous United States. 103-D-1912.

provides a step-by-step procedure that allows the user to accurately, consistently, and quickly determine the PMS. The following subsections provide a synopsis of the approach used for the various HMR reports to develop regionalized criteria.

(d) HMR 51, 52, and 53.—Generalized estimates of the PMP for the United States east of the 103d meridian for storm area sizes from 10 to 20,000 square miles and durations from 6 to 72 hours are provided in HMR 51 [35].[2] Application of these storm areal precipitation estimates for a specific drainage to obtain average basin PMS values can be obtained using the procedures in HMR 52 [18].

The PMP values obtained from HMR 51 are considered to be all-season estimates; i.e., they are the greatest values obtainable throughout the year. A companion report, HMR 53 [43], provides a seasonal variation of small area PMP for the same region based on information obtained in HMR 51.

The derivation of PMP criteria in HMR 51 initiated with the collection of severe storm observed areal precipitation data from storms occurring in or near the surrounding boundaries of the study region, section 3.3(a)(1). Each storm was maximized for moisture and transposed in accordance with section 3.3(a)(2) and 3.3(a)(3). A set of regional charts

[2]HMR 51 indicates estimates of PMP westward to the 105th meridian. Since publication of HMR 51 in 1978, the estimates of PMP between the 103d and 105th meridian have been revised and are shown in HMR 55A.

for selected storm area sizes and durations was developed, and the adjusted transposed areal precipitation from each critical storm was plotted on the appropriate chart. Also, smooth regional isohyets were analyzed on each chart. The general shape and gradients of the isohyets were patterned after several rainfall indices such as minimum envelopment of greatest daily, weekly, and monthly observed precipitation amounts [47,48,49]; 100-year precipitation analysis [22,50,51,52]; and regional distribution of maximum persisting 1,000-mbar, 12-hour dewpoints [32]. A grid was established for these charts from which DAD values of precipitation were read. The DAD values were then enveloped areally and durationally and plotted on a new set of charts from which a revised smooth regional analysis was developed, section 3.3(a)(3). Any inconsistencies were noted and addressed. Final charts of PMP were then drawn covering the study region for selected area sizes and durations. These charts show the 6-, 12-, 24-, 48-, and 72-hour PMP for 10-, 200-, 1,000-, 5,000-, 10,000-, and 20,000-square mile storm areas. A brief explanation on using the charts is also provided from which the user may quickly obtain required estimates of PMP for a specific drainage basin within the scope of the report.

Procedures are provided in HMR 52 that translate PMP values from HMR 51 to a spatially and temporally distributed estimate of the maximum storm potential for a specific drainage basin. The temporal distribution was obtained from examination of the actual occurrence of incremental precipitation from major storms of record. Such information is obtained from plots of mass rainfall curves determined for individual storms. Individual storms indicate a variety of possible temporal distributions, and HMR 52 provides general comments concerning what is and is not permitted. The actual selection of a single temporal distribution is left to the user to test what is hydrologically critical.

In the determination of the appropriate spatial distribution, an analysis of severe storm precipitation patterns led to the adoption of an elliptically shaped storm isohyetal pattern having a major to minor axis ratio of 2.5 to 1. Reduction in storm PMP is provided to account for restrictions or physical preferences as to the orientation of the PMS pattern relative to the geographic orientation of the drainage basin under study. Using DAD information from critical storms of record, the spatial distribution of precipitation, or the degree of precipitation concentration within the isohyetal pattern, was developed. While developing the spatial distribution based on storms of record, it was noted that critical areal precipitation only occurred over a restricted area and that lesser magnitude precipitation fell over both smaller and larger area sizes encompassed by the entire storm pattern. This same concept was incorporated into the spatial distribution of the PMP using within/without storm depth-area relationships to realistically distribute areal PMP precipitation. Determination of the proper spatial distribution led to the concept of residual precipitation, which is that precipitation occurring outside the PMP portion of the pattern and not considered to be PMP magnitude. Use of this important concept permits the determination of concurrent precipitation; i.e., simultaneous precipitation occurring on an adjacent drainage to that for which the PMP pattern was applied. The combined use of HMR 51 and 52 permits development of the PMS for the drainage under examination. The values obtained provide a PMF that will result from a complete hydrologic analysis.

(e) ***HMR 55A.***—Regionalized estimates used in the derivation of the all-season PMS for the region of the United States between the 103d meridian and the Continental Divide are found in HMR 55A [31]. A tentative evaluation of the maximum precipitation potential was given in a similar report published in March of 1984. Recent reviews and applications by the Bureau and NWS of the material in this 1984 publication have led to several modifications of the tentative evaluation. The revised report, HMR 55A, provides information needed for determination of the general PMS for durations up to 72 hours, for area sizes up to 20,000 square miles in basically nonorographic regions, and areas up to 5,000 square miles in the orographic influenced portions of the study area. Also, local PMS criteria are provided for durations up to 6 hours and area sizes up to 500 square miles.

Because of complexity of terrain, variety of storm types affecting the region, and lack of available storm areal precipitation data, a modified approach to the PMS determination to that used in the eastern nonorographic United States (HMR 51) was required.

Storm rainfall data [23,24,25] were examined to identify critical storms of record occurring in or near surrounding borders of the study region. These storms were classified according to the nature of the precipitation-causing mechanisms. A generalized map indicating subareas of similar topography was developed to assist in transposing available storm data throughout the study region.

Because of the variation in terrain features and difficulties encountered in transposing observed areal precipitation in such complex regions, it was decided to evaluate the maximum storm potential using a "storm separation" method. This method assumes that total precipitation from individual storms occurring in an orographic setting can be divided into two separate components for evaluation. These components are convergence, which is precipitation caused by all atmospheric processes; and orographic, which is precipitation predominantly caused by terrain influences. Using the storm separation method, the total 24-hour storm precipitation was evaluated based on individual storm relationships of observed precipitation, isohyetal patterns, and meteorological data and analysis to develop an estimate on amount of precipitation due to FAF (free atmospheric forcing) and amount due to orography.

The percentage of total 24-hour storm precipitation due to FAF was moisture maximized, transposed, and enveloped based on the principles explained in section 3.3(a). A smooth map of the FAFPMP (free atmospheric forced, probable maximum precipitation) representing the nonorographic (convergence) component of PMP over the entire region was derived for an area size of 10 square miles and a duration of 24 hours.

The next step was to determine the orographic portion of PMP and its variation over the study region. It was determined that the most reasonable method to evaluate the regional orographic variation was to determine a factor by which the FAFPMP could be multiplied to achieve total PMP. The 100-year, 24-hour precipitation frequency data [22] were used to derive this orographic factor, which was called T/C. The T simply represented the analysis of the 100-year, 24-hour precipitation that provided the combined regional variation of orographic and convergence precipitation at the 100-year level, and C represented the convergence component of T. To obtain C, maps of T (100-year, 24-hour precipitation) were examined for minimal precipitation values in defined nonorographic areas of the study region. These regions occurred in the eastern plains and broad valleys of the study region considered to contain little orographic influences. A smooth analysis indicating the convergence component of the 100-year, 24-hour precipitation was created. It was assumed that the 100-year level of precipitation and the ratio of T/C represented the orographic variation of PMP for a duration of 24 hours.

The factor T/C was further modified by the evaluation of a storm intensity factor M, which was the ratio of the most intense precipitation in a storm to the precipitation measured for the duration of interest. The M factor was obtained by evaluating such relationships in severe observed storms of record, and is used as an additional adjustment of the durational variation of dynamic forces within a storm. Smooth analysis of M values were produced over the study region. The terms that have been discussed here can be combined in the following equation:

$$\text{PMP} = \text{FAFPMP}\,[M^2(1-T/C) + T/C] \tag{4}$$

where:

PMP = total combined convergence/orographic, 24-hour probable maximum precipitation,
FAFPMP = free atmospheric forced, 24-hour probable maximum precipitation,
M = 24-hour storm intensity factor, and
T/C = 24-hour orographic precipitation factor.

The relationship shown in equation (4) produced 24-hour total PMP for specific locations assumed to represent areas of 10 square miles. The calculations were determined for a dense grid covering the study region, and a smooth analysis of the data was produced. After meteorological and topographical inconsistencies were addressed, a final map of a 24-hour, 10-square mile PMP was obtained.

Using smoothed regional analyses of 1- to 6-hour, 6- to 24-hour, and 72- to 24-hour precipitation ratios derived from observed storms of record, a 10-square mile PMP was derived for 1-, 6-, and 72-hour durations. Using relationships from severe storms of record and data supplied from adjacent PMP studies, depth-area relationships were assembled for various subregions based on topographical and meteorological variations in such parameters. Nomograms were then constructed that allowed the user to determine values of PMP for various size drainages within the HMR 55A study region and within the areal and durational limitations of the report.

Ongoing studies are being developed that will permit determination of spatial and temporal distributions of PMP for use in connection with the PMP criteria furnished in HMR 55A. Until these studies are complete, it is suggested that the interim spatial and temporal distribution criteria described in section 3.3(c)(3) be applied if required.

In the examination of PMP potential for the region, two unique storm types were identified as data sources for estimates of severe storm precipitation. These two distinct storm types are termed "general" and "local." A general storm precipitation is defined as "precipitation occurring from atmospheric/orographic processes readily defined on a synoptic scale of analysis." This type of storm can cover large areas and persist for long durations. The derivation of PMP previously described results from the analysis of precipitation due to general storm type events. Local storm precipitation is defined as "being restricted in both duration and areal extent, isolated from strong atmospheric circulations, and therefore not normally conspicuous on a synoptic scale of analysis." In the Western States, the PMS from such events, for up to 6 hours and up to 500 square miles, can often exceed the values derived from general storms for short durations and small area sizes [31,38,40]. In HMR 55A, the user must determine the PMS potential for both general and local storms. The PMS derived from each method will have to be individually evaluated in the PMF analysis and the more critical flood used for design. Because the methodology for local storm derivation is similar throughout the Western States, it will be briefly described now and referenced when other Western States HMR's are discussed [38,40].

Investigations of local PMS potential begin with identifying intense precipitation events that occurred over small areas and for short durations. Meteorological charts are then examined to select those storms that do not result from distinguishing rain-producing synoptic weather features. These storms are termed "local" in nature. Because of the sparse population in regions where local storms are effective to PMP determination and the fact that they often cover areas extremely limited in size, reports of their occurrences are rare. Fortunately, meteorological analyses of these events indicate that they can occur almost anywhere within the United States. Therefore, because the local storm data base is severely limited in quantity and quality, the use of storm transposition adjusts for this deficiency. The need for local storm evaluation is basically dependent on the comparative magnitude of PMP derived from general storm types. Where general storm type precipitation controls the level of PMP for all critical durations and area sizes required, there is little need to provide estimates of local storm PMP.

The local storms selected are adjusted to a common area size and duration, usually 1 square mile and 1 hour. In this case, point precipitation measurements are often assumed to represent an area size of 1 square mile if an actual 1-square mile analysis is unavailable. Necessary durational adjustments to observed precipitation to provide a 1-hour amount follow the depth-duration relationships described in HMR 49 [38]. Local storm precipitation representing 1-hour and 1-square mile values were moisture maximized and transposed using procedures similar to those described in sections 3.3(a)(2) and 3.3(a)(3), but modified to account for the nature and availability of associated data necessary to perform the storm maximizing procedure. Details of these modifications are found in reports covering local storm PMP determinations [31,38,40]. A regional grid of storm maximized values was established, and the data were smoothed, which resulted in an index map of a 1-hour, 1-square mile local storm PMP. Analysis of observed severe local storm precipitation revealed appropriate durational and areal relationships that were used to adjust the 1-hour, 1-square mile local PMP index values to obtain a local PMP for other durations and area sizes.

An elliptically shaped isohyetal pattern and the temporal distribution of incremental local storm PMP were also derived from examination of basic observed local storm data. From these criteria, the appropriate local PMS for use in the hydrologic analysis of the PMF was obtained.

*(f) **HMR 36.***—A detailed derivation of the regionalized general storm PMP for the Pacific Coast drainage of California is given in HMR 36 [39]. Seasonal (October through April) general storm PMP criteria for area sizes up to 5,000 square miles and durations from 6 to 72 hours are provided. For small area sizes less than 100 square miles, 1- and 3-hour values of PMP are given. The sequential distribution of incremental PMP is also provided. The magnitude of wind and temperatures occurring in conjunction with the PMS are presented for use in computing snowmelt.

The methodology used in this report is called the "orographic separation" technique. Critical assumptions used are that total PMP is derived from two separate components of precipitation (convergence and orographic), and that each component can be assessed individually and later added together to produce the complete representative level of PMP.

Data from all major storms of record are tabulated and classified as to their precipitation causing mechanisms. Intense precipitation centers located in the Central Valley of California are identified as resulting from solely convergence (no orographic influence) storms. Those storms having major centers of extreme precipitation occurring on windward slopes of the Coast and Sierra Nevada Ranges, with minimum values of corresponding precipitation in nonorographic regions, are classified as basically an orographic event. The development of the orographic separation model resulted from the meteorological combination of the causes of intense precipitation observed in both types of storms. This information led to conclusions as to the possibility of combining maximum meteorological parameters or to the imposition of various restrictions on the merging of other precipitation indices. Great care was taken so that the combination of optimum precipitation causes from the data examined did not produce unrealistically large values of total PMP.

From the information supplied by observed precipitation events in the region, the computation of two levels of convergence precipitation were necessary. The first dealt with the estimation of convergence precipitation associated with sole convergence PMP storms that occur in strictly nonorographic areas. The second computation dealt with the level of convergence precipitation to be later combined with the orographic component to provide the total PMP. Evaluation of these two types of convergence precipitation was accomplished through the smooth analysis of seasonal-durational P/M (precipitation/moisture) ratios. These ratios are the measure of the highest observed storm efficiency. The value of P is the amount of storm precipitation for a given duration, and M is the measurement of observed precipitable water associated with a storm event. Two sets of P/M ratios were developed; one based on convergence storms where instability played a minor role, and the other was determined from convergence storms where instability is an important factor.

Multiplying the appropriate P/M ratio by maximum values of moisture produced the two desired levels of convergence PMP: (1) unrestricted convergence PMP (entirely nonorographic), and (2) restricted convergence PMP (to be added later to an orographic component to provide the computed total convergence-orographic influenced PMP).

A regional smooth distribution of the restricted convergence component to the PMP was accomplished by taking into account the effect of changes in elevation, barrier influences, and adjustments for various area sizes. A single convergence restricted 6-hour, 200-square mile PMP index map for the month of January was constructed. Other monthly graphs were drawn indicating the variation of this index map with area size and duration. Values of convergence PMP that were so derived are restricted in that they have to be combined with an orographic component of PMP to produce total convergence-orographic PMP. Multiplying the restricted convergence PMP by a factor of 1.33, which is the constant ratio formed by P/M values computed from convergence storms with major instability to those with minor instability, produces unrestricted convergence PMP.

The determination of the orographic component to the total PMP, which is to be combined with the restricted convergence PMP component, was accomplished using a laminar flow orographic model. Moist air flow, which is assumed to be laminar, was lifted over a mountain barrier. Precipitation is considered to be the difference in moisture inflow above the base of the barrier and that measured as outflow above the barrier crest. The region along the windward facing slopes in the Western States presents an ideal setting for application of the model because of the broad extent of unbroken barriers and the fact that many basically orographically induced precipitation events occur in this region.

Figure 3-8 shows a view of the orographic model where resulting precipitation from multiple layers are summed to produce average precipitation over distance Y. The following equation may be used for computational purposes:

$$R = \frac{\bar{V}_1 \, \Delta p_1 \, (q_1 - q_2)}{Y} \left(\frac{1}{ge} \right) \tag{5}$$

where:

R = precipitation,
$\bar{V}_1$ = mean inflow air speed,
Δp_1 = pressure difference at inflow,
q_1 = mean specific humidity at inflow,
q_2 = mean specific humidity at outflow,
Y = distance from inflow to outflow,
g = acceleration of gravity, and
e = density of water.

The orographic model was tested and calibrated from data for extreme storms of record. A ground profile was constructed at the test location, and observed storm inflow (moisture and wind) with components of wind speeds normal to the barrier was tabulated. From this information, air streamlines were constructed and the freezing level determined so that trajectories of falling snow and rain could be computed. The model was calibrated at various locations from a set of selected storms in an attempt to replicate observed precipitation. In test storms, the convergence component of observed total precipitation was removed by subtracting the observed precipitation occurring at a nearby nonorographic location from the total observed precipitation at the test site. This was done to ensure that the model was properly calibrated to indicate precipitation due only to orographic effects.

After successful calibration of the model, the inflow parameters were adjusted to maximum values of wind and moisture, and then run to

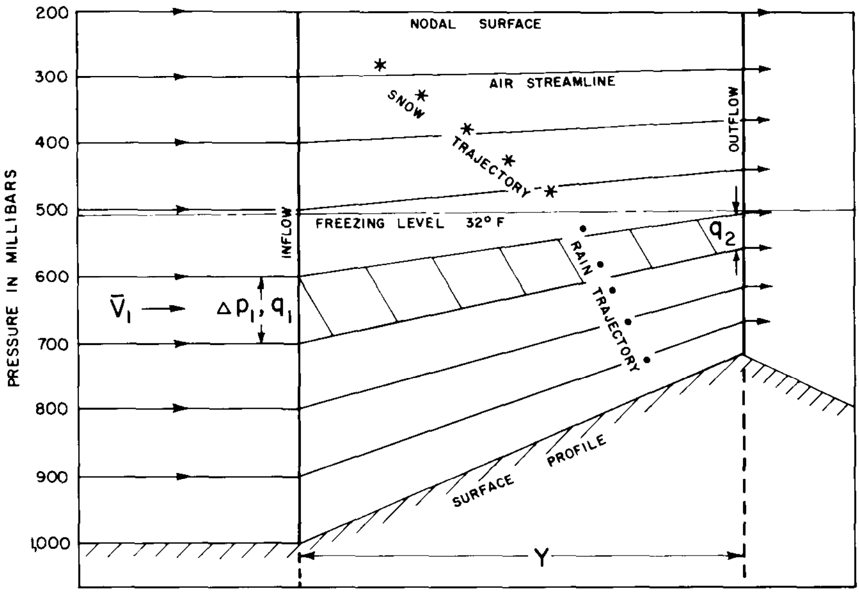


Figure 3-8.—Orographic laminar flow precipitation model. 103-D-1913.

produce maximum orographic precipitation at various locations throughout the study region. By inputing monthly variations of maximum inflow data, seasonal orographic precipitation was established.

An orographic 6-hour PMP index map for January was created for the study region. Areal extent of orographic precipitation was incorporated in the determination of a basin width factor. These indices, when combined with charts indicating the durational and seasonal variation of index orographic PMP, provided the appropriate orographic component of PMP within the scope of the study.

In nonorographic areas of the study region, it is only necessary to determine the unrestricted value of convergence PMP. In orographic areas of the study, both an unrestricted value of convergence PMP and a combined restricted component of convergence PMP and orographic PMP component producing total PMP need to be determined. The greater precipitation value should be used in calculation of the PMP.

Information regarding the sequential rearrangement of PMP increments based on similar distributions found in actual storms of record is also provided in HMR 36. The sequential distribution, section 3.3 (c), recommended by the Denver Office's Flood Section personnel that falls within the criteria established for such distributions in HMR 36, is that which should be used unless one of the other suggested distributions in HMR 36 would produce a more critical flood hydrograph. If a spatial distribution of the PMP is required, it should be patterned after an extreme storm isohyetal pattern that occurred in the drainage. The alternate and recommended distribution is one of successive subtraction of subbasin volumetric precipitation, section 3.3 (c)(3).

Criteria for calculation of associated winds and temperatures with the occurrence of the PMS event for snowmelt considerations can also be determined from HMR 36. It is recommended that the alternate snowflood criteria discussed in section 4.3 of chapter 4 be considered before application of the criteria given in HMR 36.

The development of PMP criteria in HMR 36 was predominately based on the meteorological analyses of historical severe storms of record. Details of these analyses are reported in HMR 37 [53].

*(g) **HMR 43.***—Regionalized estimates of PMP for the Northwest United States, Columbia River Basin, and Coastal Drainages of Washington and Oregon are provided in HMR 43 [40]. General storm seasonal (October through June) PMP estimates are provided for area sizes from 10 to 5,000 square miles west of the Cascade Divide and for 10- to 1,000-square mile areas east of the Cascade Divide to the Continental Divide. General storm durational PMP is provided up to 72 hours. For the region of the study area east of the Cascade Divide, a summer local thunderstorm PMP is provided for area sizes up to 550 square miles and for durations up to 6 hours. Similar to the procedures in HMR 36, suggested sequential rearrangement of the general storm PMP is provided along with associate temperature and wind sequences for use in determination of snowmelt contributions to the flood hydrograph. Also, derivation of local thunderstorm criteria is similar to that previously described in the discussion of HMR 55A. Much of the methodology used to develop the PMP for the Northwest States is similar to the procedures given in HMR 36 and will not be reviewed in detail here.

Major storms of record affecting the study region are identified for three separate major subregions: (1) storms occurring west of the Cascades, (2) storms in the central interior of the Columbia Basin, and (3) storms located near the Continental Divide. From the meteorological analysis of these storms, lack of analyzed storm data, and the difficulty of transposing individual storms in the region due to the complex terrain, the decision was made to separately compute components (convergence-orographic) of total storm precipitation to develop the PMP potential for the region. The method chosen was a modified application of the orographic separation technique previously developed in HMR 36.

Investigations into the determination of the convergence component of total storm PMP began with the calculation of separate P/M ratios based on those convergence type storms indicating minimal or maximum associated instability in their storm process. Storms examined were those occurring in the nonorographic areas along the west coast of the study region. These ratios, used as a measure of storm efficiency, were calculated for the month of January from severe observed 24-hour precipitation and associated moisture at the time of the storm event. A comparison of P/M ratios associated with maximum convergence storm conditions (unrestricted) to ratios associated with maximum orographic storm conditions (restricted) indicated a constant relationship of 1.23. Using the restricted P/M result, a smooth field of P/M ratios (storm efficiency) was created based on such ratios calculated along the west coast and distributed inland based on variations reflected on seasonal charts of maximum moisture. Maps of highest monthly dewpoint data (moisture index) had been tabulated, enveloped seasonally, adjusted to a 1,000-mbar surface level, and smoothed regionally. Multiplication of restricted P/M ratios by maximum moisture produced a regionalized map of January convergence 24-hour, 10-square mile, 1,000-mbar PMP (restricted) to be combined with an orographic component of PMP. Monthly graphs of maximum observed precipitation data were constructed to distribute seasonally the January convergence PMP. Index maps of seasonal 24-hour, 10-square mile, 1,000-mbar orographic (restricted) storm convergence PMP were prepared. Monthly charts of 6- to 24-hour and 24- to 72-hour precipitation ratios were prepared to provide convergence precipitation for other durations based on ratios developed using data from observed storms of record. Depth-area relationships were developed to adjust 10-square mile convergence PMP for various area sizes, up to 5,000 square miles west or 1,000 square miles east of the Cascade Divide. Adjustments for terrain features (elevation-barrier/depletion-stimulation effects) associated with the individual basin location within the study region were developed to adjust 1,000-mbar convergence PMP to surface elevation. Combination of these various adjustments, displayed in figures or tables, allow the user to quickly adjust convergence (restricted) PMP to specific locations throughout the study region. Unrestricted convergence PMP for locations west of the Continental Divide is obtained by multiplying restricted convergence PMP by a factor of 1.23.

The orographic component to PMP was derived using the orographic separation model developed in preparing HMR 36. Model tests were made at four widely separated sites in the HMR 43 study region. Observed orographic precipitation occurring at these sites was used to calibrate the model for use in these particular subregions of the study, similar to the method used in developing the model adjustment for HMR 36. Seasonal values of maximum windspeeds and moisture were used in conjunction with the calibrated orographic model to develop corresponding seasonal maximum values of precipitation, see HMR 36 for details of model adjustments.

Because optimum moisture-inflow direction and orientation of slopes varied in the region east of the Cascade Crest, generalized terrain profiles for the entire study region, orientated in different directions, were established and model maximum precipitation values calculated for various inflow directions. Envelopes of greatest computed orographic precipitation from various combinations of slope or inflow orientations were plotted for each region. Relationships were developed to adjust envelope values to specific basin critical inflow and slope direction.

From the above analysis, a smooth regional 6-hour, 10-square mile orographic PMP index map was developed. Seasonal variations of PMP values were obtained by examination of model computations and seasonal variation of observed precipitation. Durational variation of orographic PMP was obtained from model calculations related in time at various test locations. An areal size variation in orographic PMP was established based on depth-area relationships from several observed storms occurring in the region.

To compute total storm PMP, components of orographic PMP and orographic convergence (restricted) PMP are added together. In addition to total storm component calculations of PMP, it is necessary to compute a convergence storm (unrestricted) level of PMP for the more nonorographic areas located west of the Cascade Divide. This convergence precipitation, which is not to be combined with the orographic component, is obtained by multiplying restricted convergence PMP by a factor of 1.23. The two PMS values should be determined and the PMF hydrographs developed. Each hydrograph should then be routed through the structure to determine which hydrograph is critical for design purposes.

(h) HMR 49.—Regionalized estimates of seasonal general-storm PMP for the Colorado River and Great Basin drainages are provided in HMR 49 [38]. General-storm PMP criteria covers area sizes up to 5,000 square miles for durations of 6 to 72 hours. In addition to the above drainages, all-season local-storm PMP is provided for all of California. Local-storm PMP covers area sizes between 1 and 500 square miles and for durations from 15 minutes to 6 hours. Derivation of local-storm PMP follows the same procedure previously discussed in HMR 55A.

The methodology applied in the determination of general-storm PMP is to derive separate estimates of orographic and convergence PMP. These components are later added to find the total general-storm PMP. The method is somewhat similar to the method for developing the general-storm PMP for the Northwest States and California, HMR 43 and HMR 36, respectively. Observed precipitation data, in defined nonorographic subareas of the study region, were moisture maximized and enveloped. The storm maximization process necessitated the need for updated charts of maximum 12-hour persisting 1,000-mbar dewpoints. Seasonal distributions of maximum moisture and maximum observed precipitation were used as guides to interpolate between locations of nonorographic moisture-maximized precipitation. Seasonal index maps of 1,000-mbar convergence PMP for 24 hours and 10 square miles were established for the study region. Adjustment to convergence PMP at 1,000-mbar for the effects of elevation and barrier were incorporated. Depth-area and depth-duration relationships were developed to obtain convergence PMP for other area sizes and durations. These relationships were based on DAD relationships from observed nonorographic storms occurring in both the study region and in the plains of the Central States. Resulting convergence PMP values were in general agreement with similar convergence component estimates derived for adjacent regions to the study area, HMR 36 and 43.

In calculating the orographic component of PMP for the HMR 49 region, the orographic precipitation laminar flow model developed in HMR 36 was not used. Based on an understanding of the greater convective activity of severe storms occurring in the region and the complexity of terrain features, only limited use of the orographic inflow model could be used satisfactorily. The alternate procedure developed was based on observed storm precipitation and its variation due to terrain effects.

A first approximation to the estimation of an orographic component of PMP involved the use of 100-year, 24-hour precipitation data obtained from NOAA Atlas 2 [22]. It was assumed that the regional complexity of the 100-year precipitation was mainly due to orographic influences, and the 100-year level of precipitation would most likely represent that for the level of PMP with little additional modification.

In the nonorographic regions of the study region, frequency values from NOAA Atlas 2 were assumed to represent precipitation from entirely convergence storm mechanisms after modifications for surface level and barrier effects were applied to adjust frequency values to a common surface for analysis. Total 100-year, 24-hour precipitation was expressed as a percent of this regional 100-year, 24-hour convergence component. Multiplication of these percentages by the convergence component of PMP resulted in the first approximation of orographic PMP for the study area. This method of orographic component evaluation assumes that the relationship of component convergence to orographic 24-hour PMP is similar to that derived using 100-year, 24-hour precipitation. This first approximation was modified based on several meteorological parameters. Examples of these modifications would be the adjustment of the first approximation to orographic precipitation by the examination of precipitation occurring along terrain profiles as analyzed from charts of observed storm precipitation and mean annual and seasonal precipitation. Incorporation of these adjustments and those from other indices resulted in the adoption of a final 10-square mile, 24-hour orographic PMP index map. A variation of orographic PMP with basin size was developed using depth-area relationships from major storms occurring in the region. The durational variation of orographic PMP was adopted from examinations of the durational variations noted in maximum winds and moisture affecting the region. The adopted durational variation was checked and adjusted by comparison of durational precipitation associated with severe storms occurring in the region and durational ratios formed by combining maximum values of 6-, 24-, and 72-hour observed storm precipitation. Seasonal variation of orographic component PMP was accomplished by tying into seasonal variations obtained from HMR 36 and 43 near boundaries of the study region and by using various seasonal related precipitation indices within the study region. Examples of the indices examined are: (1) seasonal variation of observed maximum precipitation at various stations within study region, (2) seasonal variation of maximum moisture and winds, and (3) seasonal computations of precipitation expressed by the orographic precipitation model described in HMR 36 and 43.

By using convergence and orographic index charts and other figures, and graphs or tables to describe the areal, durational, and seasonal variations of index values of PMP, separate evaluations of convergence and orographic component PMP are established. These components must be added together to provide total PMP for desired individual drainages bounded by the limiting scope of HMR 49.

Temporal and spatial distributions of general-storm PMP are not provided in HMR 49. It is Bureau practice to use the procedure described in section 3.3(c)(3) to sequence increments of total precipitation as well as to obtain the spatial distribution of general-storm PMP.

Supporting data upon which regionalized PMP criteria were developed are also shown in HMR 50 [33]. Discussions of individual general and local storms important to the development of criteria for estimating PMP in HMR 49 are presented as well as the derivation of seasonal maximum moisture charts (dewpoints).

(i) Summary.—The methodologies briefly describing the derivation of the PMS from an individual drainage study or through regionalized analyses have been presented. Use of the individual drainage approach should be left to trained hydrometeorologists to develop. The steps for determining the PMS as set forth in various HMR's are in simple, easy-to-follow, step-wise procedures. Several of these procedures have been adapted for computerized analyses, and these computer programs are available from the Flood Section at the Bureau's Denver Office.

In the development of individual drainage estimates of PMP, the resulting values should be checked for consistency, and the general magnitude achieved. Samples of such checks generally consist of comparisons: (1) with previous estimates derived for drainages or regions in surrounding area, (2) with observed storm data of record, and (3) with extreme values of precipitation derived by statistical means. For PMP obtained using the HMR series, evaluation of the latter two comparisons have been completed for most of the United States [34]. Additionally, most regionalized reports internally address some type of evaluation as to the level of PMP obtained.

Revision and refinements of PMS procedures, techniques, and methodologies are an ongoing process. As new severe storms occur and are recorded, or better theories developed, current procedures and reports will be evaluated for their adequacy. Where deficiencies exist, revisions and refinements will be made and documented in subsequent reports.

3.4 Distributions of PMS

In the development of the general PMS based on individual drainage estimates or regionalized studies, temporal and spatial distributions of the design precipitation are often required. Following the general comments made in sections 3.3(c)(1) and 3.3(c)(2), a variety of methods exist, each producing an array of possible distributions. The combined efforts of Bureau regional and Denver Office hydrologists have resulted in some basic concepts that can be generally applied to derive appropriate PMS distributions keeping within meteorological criteria and the desire to achieve critical conditions for PMF hydrograph analysis.

(a) Temporal.—From examination of individual drainage and regionalized storm criteria combined with various hydrological tests, the following steps were derived to provide a single PMS temporal distribution that has been adopted for use throughout the United States:

Step 1. Determine unit duration for unit hydrograph analysis as discussed in chapter 4.

Step 2. Obtain increments of unit duration precipitation for total storm period from appropriate individual drainage or regionalized studies from the smooth PMP depth-duration curve for the basin or subbasins.

Step 3. Place greatest amount of unit duration precipitation at the two-thirds position in the total storm period.

Step 4. Place second and third greatest unit duration precipitation before and adjacent to the greatest increment in descending order.

Step 5. Place fourth greatest incremental precipitation after and adjacent to the greatest incremental amount.

Step 6. Place remaining unit duration incremental precipitation amounts in a similar fashion to the placement of the four greatest precipitation increments.

Figure 3-9 illustrates this preferred distribution as applied to a 24-hour storm of 1-hour duration. Longer duration storms would use a similar distribution.

(b) Spatial.—Various methods of areally distributing PMP have been developed over the years. Caution should be used in the application of these methods, and their incorporation should generally be determined by trained hydrometeorologists. For obtaining the spatial distribution of regionalized derived general-storm PMP for the United States, the following criteria should be applied:

(1) For nonorographic regions of the Eastern States covered by HMR 51 and portions of HMR 55A, the spatial distribution criteria shown in HMR 52 is deemed applicable.

(2) For the remaining regions of the United States, an interim technique called "successive subtraction of subbasin PMP volumes" should be applied until adequate regionalized spatial distributions of PMP are developed. Because hydrologic analysis is performed from averages of

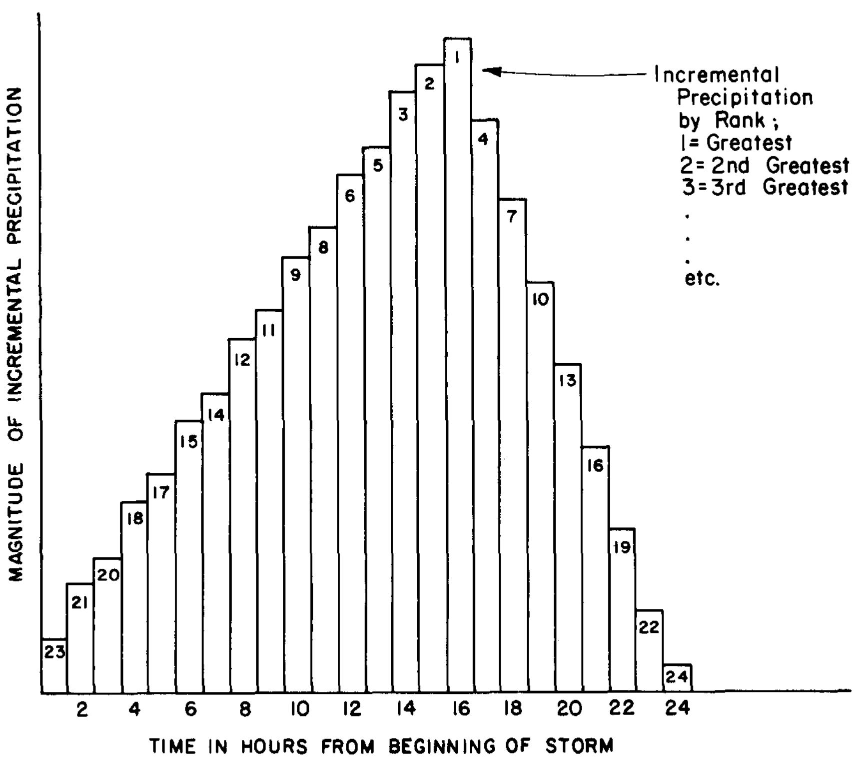


Figure 3-9.—Sequential arrangement of PMP increments for a 24-hour storm. 103-D-1914.

meteorological and hydrological parameters over basins or subbasins, as discussed chapter 4, it is only necessary to determine the averages of design storm precipitation calculated for the total drainage, and that required for individual or joint subbasin analysis. When the PMP is determined for a certain basin or combination of subbasins, it is tantamount to saying the storm is centered over that basin or subbasin combination. Using the above criteria, the spatial distribution of PMP by the "successive subtraction of subbasin PMP volumes" technique can be applied. The basic equation for this technique is:

$$P_T A_T = P_1 A_1 + P_2 A_2 + P_3 A_3 + \ldots + P_N A_N \qquad (6)$$

Equation (6) equates precipitation volume over an area, A_T, to the summation of precipitation volumes over subareas A_1, A_2, A_3, ... A_N. The precipitation volume, PA, is the average precipitation, P, times the area, A, over which the precipitation falls. Subscript N identifies the individual subbasins, and subscript T represents total area of the drainage. Since the purpose of spatial distribution is to provide information for peak discharge calculations, tests of various subbasin centerings of the PMP should be evaluated.

Details of this technique are best described in the following example: Assume a basin with total area size A_T is broken into three separate subbasins of area sizes A_1, A_2, and A_3 for hydrologic analysis. It is necessary to assess the spatial distribution of the PMP determined for the entire drainage area from the appropriate orographic area report, HMR 36, 43, 49, or 55A. The first step consists of centering the PMP in subbasin 1, and then determining concurrent precipitation in subbasins 2 and 3.

Step 1. Find the average precipitation P_1 over area A_1, where the PMP is assumed centered. The P_1 is directly obtained from the appropriate HMR for area size A_1 and the duration of interest. The result is the average depth of PMP; i.e., the P_1 centered in subbasin 1.

Step 2. Find concurrent average precipitation P_2 when PMS is centered in subbasin 1:

$$P_2 = \frac{P_{1,2}\, A_{1,2} - P_1 A_1}{A_2}$$

The value for $P_{1,2} A_{1,2}$ is obtained directly from the appropriate HMR for the combined area size $A_1 + A_2$, and $P_1 A_1$ is the value determined in step 1 for subbasin 1.

Step 3. Find the concurrent average precipitation P_3 when PMP is centered in subbasin 1 with concurrent precipitation in subbasin 2:

$$P_3 = \frac{P_{1,2,3}\, A_{1,2,3} - P_{1,2}\, A_{1,2}}{A_3}$$

The value for $P_{1,2,3} A_{1,2,3}$ is obtained directly from appropriate HMR for combined area size $A_1 + A_2 + A_3$, and $P_{1,2} A_{1,2}$ is the value calculated in step 2 for the PMP volume centered in subbasins 1 and 2.

These computations are repeated for each durational increment of precipitation necessary to describe the spatial distribution for the entire storm period. The entire procedure is evaluated for different storm centerings; i.e., PMP centered over subbasin 2 and then subbasin 3. The various spatial distributions are then tested hydrologically, and the distribution resulting in the critical peak discharge PMF is chosen for design.

Studies updating and developing regionalized temporal and spatial general PMS distributions in the United States for HMR 43 and 55A are currently underway. Similar information for other regions of the Western States will be addressed in future reports. Until such information becomes available, the temporal and spatial derivation procedures outlined in this chapter should be applied.

Chapter 4
FLOOD HYDROGRAPH DETERMINATIONS

4.1 Flood Runoff From Rainfall

(a) General.—Chapter 3 presented the criteria and procedures for the determination of PMS amounts that could be experienced over a drainage basin. This chapter concerns the processes involved with converting rainfall and/or snowmelt into a hydrograph representing the upper level or maximum flood runoff that a drainage basin might reasonably be expected to produce. As previously mentioned, the unit hydrograph approach is the basic tool or "model" to convert rainfall to runoff after abstracting suitable infiltration losses. However, it should be noted that there are a number of other techniques available for making this conversion, including highly complex computerized watershed models.

In 1932, Leroy K. Sherman [5][1] initially proposed the unit hydrograph approach to convert rainfall occurring over a drainage basin to flood runoff from that basin. Sherman's approach, which was formally presented in the April 7, 1932, issue of *Engineering News Record*, has undergone considerable refinement over the years. The advent of high speed electronic computers has led a number of hydrologists to devise approaches using complex watershed models, as alternatives to the unit hydrograph model, to predict a drainage basin's runoff response to rainfall. Many of these watershed models are an appropriate basis for simulating a continuous series of runoff responses to normal precipitation events. However, in the Bureau of Reclamation's application, the primary interest is in simulating a basin's runoff response to extreme rainfall events. Because these complex watershed models generally require extensive calibration to adequately represent a drainage basin's physical properties, considerable effort must be expended in the field and office in acquisition of data relative to these properties. In the final analysis, the relative "goodness" of an approach is measured by how well that approach reproduces actual recorded flood events. Comparative studies have indicated that both approaches are able to satisfactorily reproduce these events with neither one being notably superior to the other. Accordingly, the Bureau has, over the years, retained the unit hydrograph approach because of its simplicity, reliability, and the relatively low costs associated with its application to flood hydrology studies.

(b) Basic Unit Hydrograph Concept.—The unit hydrograph may be defined as "the discharge hydrograph resulting from 1 inch of direct runoff generated uniformly over the tributary area at a uniform rate during a specified time period." The concept of the unit hydrograph theory follows five basic assumptions [2]:

[1]Numbers in brackets refer to entries in the Bibliography.

1. Effective rainfall is distributed uniformly over the entire drainage basin, and is the amount of rainfall available for runoff after infiltration, surface ponding, and other losses have been deducted.

2. The effective rainfall is uniformly distributed over a specific period of time, which is referred to as the "unit duration."

3. The time duration of the base of the surface runoff hydrograph resulting from the effective rainfall of unit duration is constant.

4. Ordinates of surface runoff hydrograph are proportional to amount of unit effective rainfall.

5. The surface runoff hydrograph for a given drainage basin reflects all of the unique physical characteristics of the drainage basin.

Considering the above basic assumptions, it appears that the unit hydrograph concept represents the modeling of the rainfall-runoff process as a linear system. The fact that the rainfall-runoff process is actually nonlinear is one of the acknowledged shortcomings of the concept. However, if properly applied, the concept provides entirely satisfactory results for developing flood hydrographs. Proper application of the concept is treated in subsequent parts of this chapter.

The basic concept of the unit hydrograph theory [9] can be explained by considering a situation where a storm of 1-hour duration produces rainfall at a constant rate over that duration, and occurs uniformly over the entire drainage basin above a recording stream-gauging station. Assume the rate at which the rain is falling is such that 1 inch of surface runoff results, and that this runoff flows to tributary watercourses arriving eventually at a stream-gauging station. The runoff at the gauging station will be recorded as a hydrograph representing the temporal distribution of runoff resulting from 1 inch of "rainfall excess" occurring in 1 hour. The recorded hydrograph is then said to be the "1-hour unit hydrograph" for the basin contributing runoff at the gauging station, and the unit hydrograph is said to have a "unit duration" of 1 hour. The significance of unit duration will be discussed later in this chapter.

Now consider the situation where the "rainfall excess" was 2 inches in a 1-hour period. The unit hydrograph theory assumes that the 1-hour hydrograph ordinates are proportional to the rainfall excess. It follows that the runoff hydrograph at the gauging station resulting from 2 inches of rainfall excess can be predicted by multiplying each of the 1-hour unit hydrograph ordinates by a factor of 2. Naturally, this is true for any multiple or fraction of an inch of rainfall excess, as shown on figure 4-1.

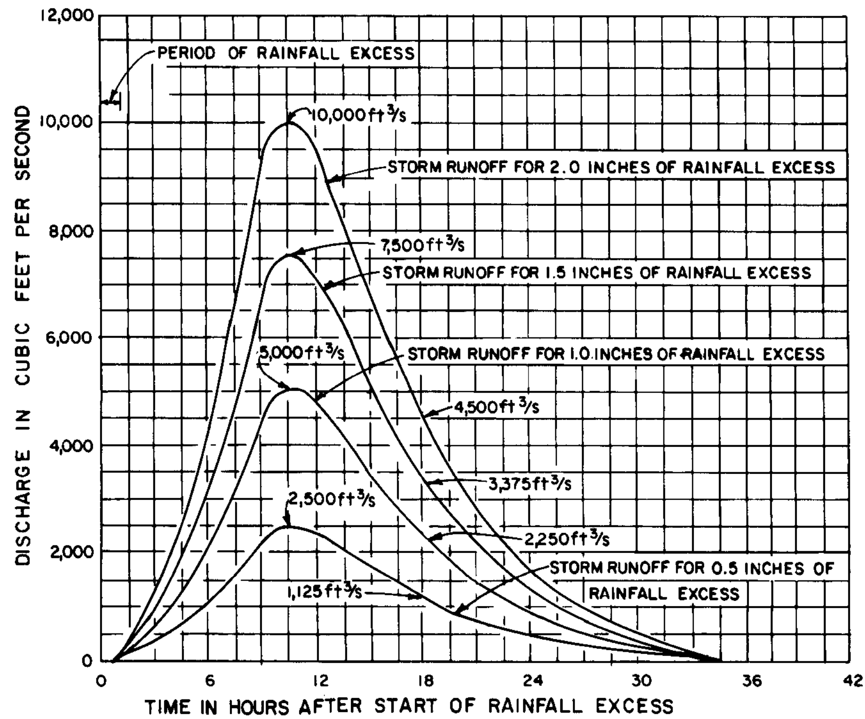


Figure 4-1.—Unit hydrograph principles. 103-D-1848.

The discussion to this point has basically considered an isolated rainfall event sustained for a period equal to the unit duration of the unit hydrograph. Unfortunately, nature does not usually behave in such a simplistic manner. The severe storms that occasionally visit every drainage basin, regardless of location, are both longer than the unit duration and more varied in intensity from one "unit" period to another. Figure 4-2 illustrates the manner in which the unit hydrograph approach takes severe storms into consideration.

Figure 4-2 shows that each of the five increments of precipitation excess results in an incremental runoff hydrograph, as shown by the 0.4 to 1.5-inch curves at the bottom of the figure. Each incremental hydrograph is determined by multiplying the increments of rainfall excess by the drainage basin's unit hydrograph ordinates. The total runoff from the complex rainfall event, only the excess is shown on figure 4-2, can be determined by adding the ordinates of each runoff hydrograph at discrete time intervals. These time intervals are usually equal to the unit duration of the unit hydrograph. The resulting runoff hydrograph can be drawn by graphically connecting these ordinate points in a curvilinear fashion.

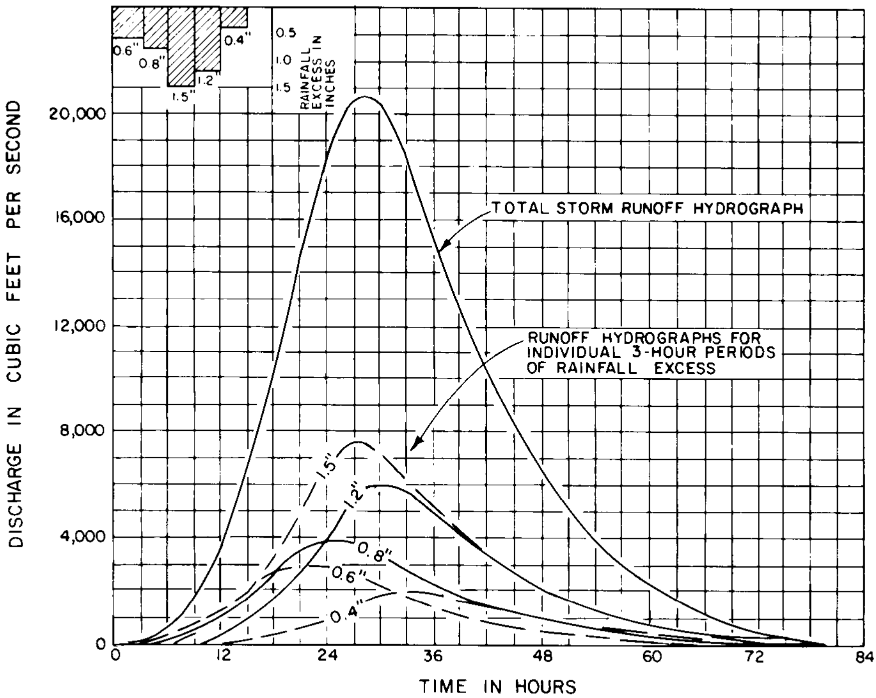


Figure 4-2.—Unit hydrograph application. 103-D-1849.

In actual practice, the hydrologic engineer is usually faced with the problem of providing a flood hydrograph for design purposes at a location where no streamflow data have been obtained. The surface runoff portions of flood hydrographs at these sites are developed using hypothetical rainfall amounts that could occur over the drainage basin, appropriate infiltration loss rates, and a synthetic unit hydrograph. The total flood hydrograph is determined by adding appropriate base flow or snowmelt flow to the surface rainfall-runoff hydrograph.

Synthetic unit hydrographs are developed from parameters representing the salient features of the rainfall-runoff phenomena found by reconstituting observed flood events on similar drainage basins. In general, synthetic unit hydrographs are satisfactory when generated for drainages up to about 500 square miles. Larger basins should be divided into subbasins of about 500 square miles, and a separate synthetic unit hydrograph generated for each subbasin, which are then routed and combined to form the hydrograph for the total basin. Reconstitution of observed events generally provide two significant features or items of information: (1) an indication of infiltration rates to be expected with the composite soil types present in the drainage basin, and (2) a unit hydrograph for each basin analyzed. Associated with each unit hydrograph are two features that are used to determine synthetic unit hydrographs for ungauged drainage basins: (1) a relationship representing the variation of runoff over time, and (2) the time that the rise in runoff lags the rainfall causing the rise. The latter feature is called the "unit hydrograph lag time."

(c) Unit Hydrograph Lag Time.—Over the years, numerous observed floods have ben reconstituted using the unit hydrograph approach. A graphical example of flood hydrograph reconstitution is shown on figure 4-3. Analysis of these reconstitution results has led to the conclusion that a unit hydrograph's lag time varied as a function of certain measurable basin parameters. Lag time was originally defined in 1936 by Horner and Flynt [54] as the ". . . time difference in phase between salient features of the rainfall and runoff rate curves." In 1938, Snyder [55] developed the following relationship for lag time based on studies of basins in the Appalachian Mountain region:

$$L_g = C_t\,(LL_{ca})^{0.3} \tag{1}$$

where:

L_g = lag time, in hours;
C_t = a coefficient that varies from 1.8 to 2.2;
L = length of longest watercourse, in miles; and
L_{ca} = length along L to a point opposite centroid of drainage basin, in miles.

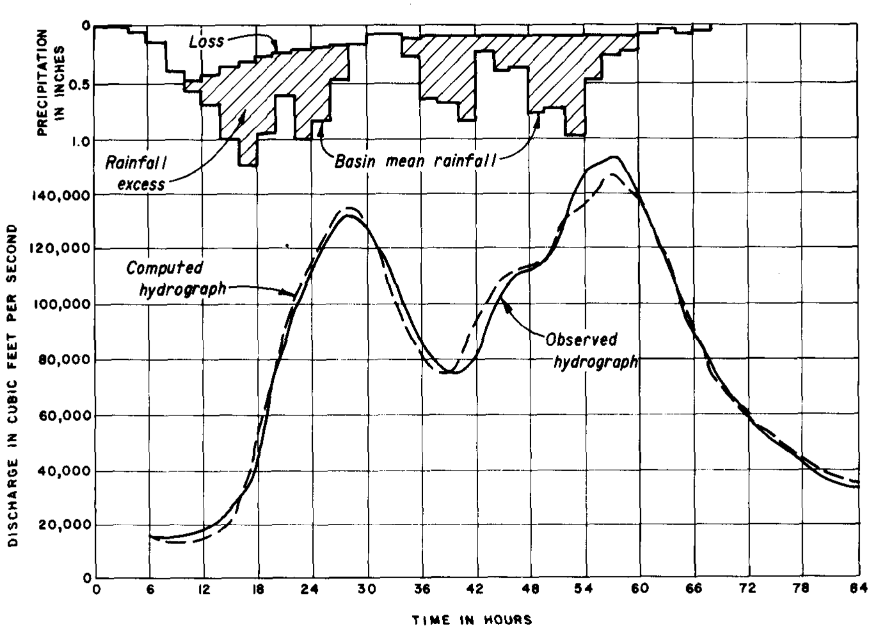


Figure 4-3.—Typical flood hydrograph reconstitution. 103-D-1915.

In 1944, the Los Angeles District of the Corps of Engineers [56,57] introduced the slope of the longest watercourse into Snyder's equation, which resulted in the following general relationship between lag time and measurable basin parameters:

$$L_g = C_t \left(\frac{LL_{ca}}{S^{0.5}} \right)^N \tag{2}$$

where:

L_g = lag time, in hours;
C_t = a constant;
L = length of longest watercourse from point of concentration to boundary of drainage basin, in miles (point of concentration is that location on watercourse where a hydrograph is desired);
L_{ca} = length along L from point of concentration to a point opposite centroid of drainage basin, in miles;
S = overall slope of L, in feet per mile; and
N = an exponent (the Bureau currently uses N = 0.33).

The index of equation (2), $LL_{ca}/S^{0.5}$, formed the basis for procedures used in the Bureau's 1952 publication *Unitgraph Procedures* [9].

Recent analyses of unit hydrograph data for many drainage basins throughout the United States, as they have become available, have led Bureau hydrologic engineers to the conclusion that the value of the exponent N can be taken as 0.33 regardless of the regional location of a particular drainage basin. Additional detailed analyses of these data have led these same engineers to conclude that C_t can be expressed as 26 times the average Manning's n value representing the hydraulic characteristics of a drainage basin's drainage network. This average n value is identified as K_n in subsequent considerations of lag time in this manual, therefore, $C_t = 26K_n$. It should be emphasized that K_n is primarily a function of the magnitude of discharge and will normally decrease with increasing discharge. This "nonlinear" condition is inconsistent with the "linear" basic assumptions of the unit hydrograph concept. Accordingly, the K_n value should be set to reflect hydraulic conditions that would exist in extreme flood conditions.

Current Bureau practice uses two definitions of unit hydrograph lag time, depending on the technique used, that are somewhat different than originally proposed by Horner and Flynt [54]. Lag time definitions, depending on the technique for synthetic unit hydrograph development being used, are as follows:

(1) *Dimensionless unit hydrograph technique* [9].—Lag time is the time from center of unit rainfall excess to the time that 50 percent of the

volume of unit runoff from the drainage basin has passed the concentration point. This concept is displayed graphically on figure 4-4.

(2) *S-graph technique* [56,57].—Lag time is the time from the start of a continuous series of unit rainfall excess increments to the time when the resulting runoff hydrograph reaches 50 percent of the ultimate discharge. The ultimate discharge is an equilibrium rate achieved at the time when the entire drainage basin is contributing runoff at the concentration point from the continuous series of unit ranfall excess increments. This relationship is shown graphically on figure 4-5.

(d) Temporal Distribution of Unit Runoff.—The determination of a basin's unit hydrograph lag time is only half the information required for developing a synthetic unit hydrograph. The remaining half is the means by which the runoff from the unit effective rainfall is distributed over time. This temporal distribution is accomplished by using a dimensionless form of an observed unit hydrograph for a hydrologically similar drainage basin. By using a dimensionless form, differences in drainage basin size and variations in unit hydrograph lag time and unit duration are automatically taken into consideration. There are currently two methods used by the Bureau that utilize the dimensionless form of the unit hydrograph: (1) Dimensionless Unit Hydrograph Technique, and (2) S-Graph Technique.

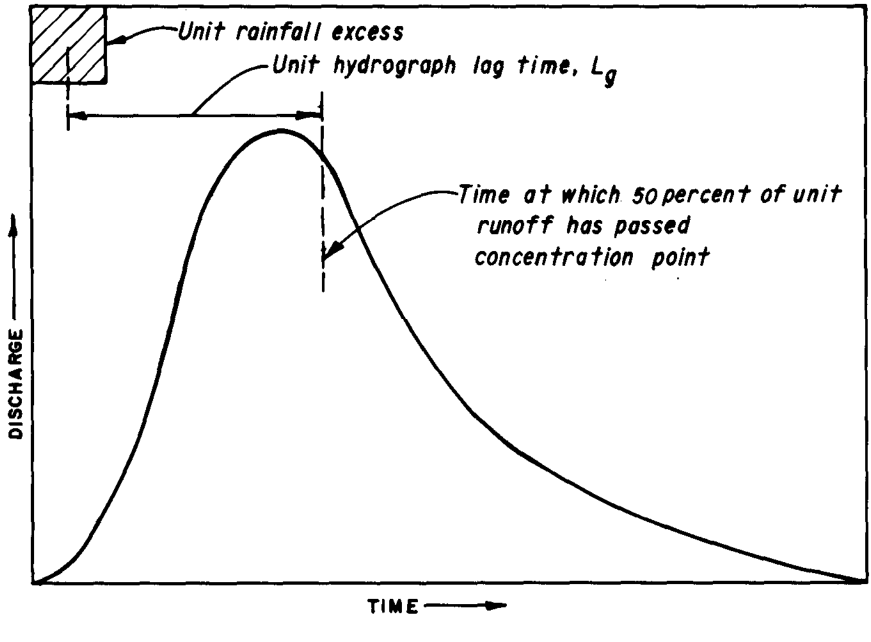


Figure 4-4.—Dimensionless unit hydrograph lag time. 103-D-1916.

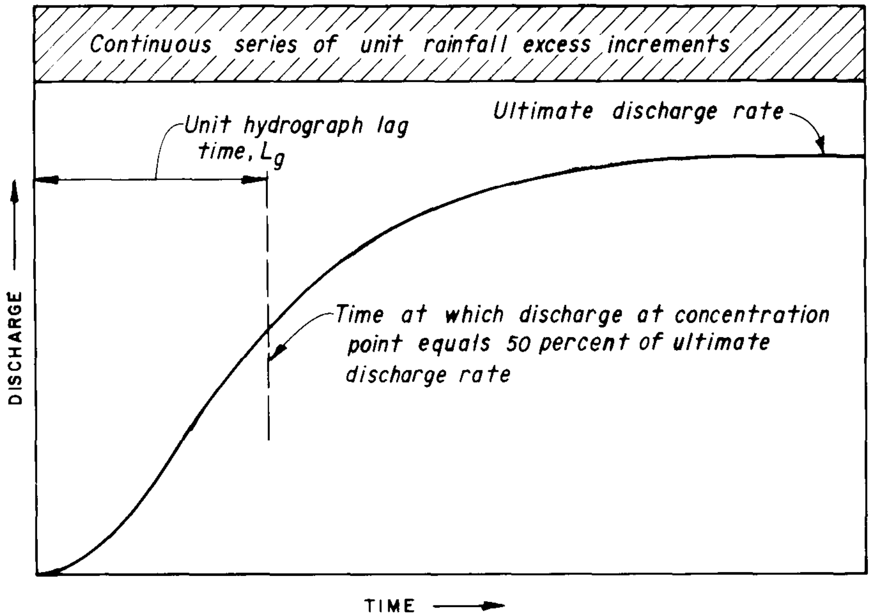


Figure 4-5.—S-Graph lag time. 103-D-1917.

(1) *Dimensionless unit hydrograph technique* [9].—Unit hydrographs developed from recorded flood events are converted to dimensionless form for use in synthetic unit hydrograph development as follows:

a. The time base (abscissa) is expressed in terms of time t as a percentage of the lag time L_g plus 50 percent of the unit rainfall duration D. Mathematically, this may be expressed as

$$\frac{100t}{L_g + 0.5D}$$

Increments of time t are equal to the unit duration of the unit hydrograph derived from the recorded flood event.

b. Dimensionless discharge ordinates q are expressed in terms of the product of the recorded flood unit hydrograph ordinate Q, in cubic feet per second, and lag time L_g plus 50 percent of the unit rainfall duration $L_g + 0.5D$ divided by the unit runoff volume for the basin V', in 1-day cubic feet per second. Expressed mathematically, this relationship is:

$$q = \frac{Q\,(L_g + 0.5D)}{V'} \qquad (3)$$

(2) *S-graph technique* [56,57].—Unit hydrographs developed from recorded events are converted to dimensionless form as follows:

a. A summation hydrograph is initially developed by algebraically adding the ordinates of a continuous series of identical unit hydrographs, each successively out of phase by one unit period. The lag time for this particular technique is determined by reading from the plotted summation hydrograph, the elapsed time from the beginning of rainfall to the time when 50 percent of the ultimate discharge is reached.

b. The dimensionless hydrograph is then developed from the summation hydrograph by converting the time base (abscissa) to time in percent of lag time and converting the ordinate values to discharge as a percent of the ultimate discharge.

(e) Development of Synthetic Unit Hydrographs.—In chapter 2, considerable attention was given to the specific observations that should be made during a field inspection of a drainage basin. Observations made relative to the basin's drainage network or hydraulic system form the primary basis for establishing an appropriate K_n value [10] to be used in estimating the synthetic unit hydrograph lag time. In assigning a K_n value for a particular basin, consideration should also be given to K_n values developed from analyses of observed flood hydrographs for basins that are similar with respect to general topography, to channel and flood plain characteristics, and to drainage network density.

Once the value of K_n has been estimated, the length of the longest watercourse, L, and the length along the longest watercourse to a point opposite the centroid of the drainage basin, L_{ca}, are measured. A suitable topographic map such as a USGS quadrangle map is usually used for these measurements. The slope of the longest watercourse, S, is also determined using contour data from the topographic map. The drainage basin's physical parameters K_n, L, L_{ca}, and S are then entered into the general lag equation (1):

$$L_g = 26\ K_n \left(\frac{LL_{ca}}{S^{0.5}} \right)^{0.33} \tag{4}$$

where:

L_g = lag time, in hours;
L = distance of longest watercourse, in miles;
L_{ca} = distance from gauging station to a point opposite centroid of drainage basin, in miles;
S = overall slope of L measured from gauging station or point of interest to drainage basin divide, in feet per mile; and
K_n = a trial value based on an estimate of the weighted, by stream length, average Manning's n value for the principal watercourses in the drainage basin.

Equation (4) yields the synthetic unit hydrograph lag time in hours. The results of applying this equation are considered adequate for either the dimensionless unit hydrograph or S-graph approach:

(Lag Time)S-Graph $\approx$ (Lag time + Semiduration)Dimensionless Graph

To aid in determining an appropriate lag time, many flood hydrograph reconstitutions have been examined. These reconstitutions represent flood runoff from natural basins throughout the conterminous United States west of the Mississippi River and from urbanized basins for several locations throughout the States. Data for urbanized basins are included in this manual because of the increased interest in the flood hydrology of such areas, particularly with respect to the impact on runoff from various levels of development.

As a result of the examination of these reconstitutions, 162 flood hydrographs considered representative of surface runoff from rainfall events were selected for analysis relative to regionalized trends in the lag time relationships and the time versus variation of discharge relationships. Those hydrographs not included were considered to represent either interflow runoff or runoff that included significant contributions from snowmelt. The 162 hydrographs were then segregated on a regional and topographic basis, as shown on figures 4-6 through 4-11. The supporting data for these figures are listed in tables 4-1 through 4-6. These tables include the station index number, station name and location, drainage area (in some cases, only the area contributing to flood runoff), basin factor $LL_{ca}/S,^{0.5}$ unit hydrograph lag time determined from the flood hydrograph reconstitution, computed K_n value, and the C_t constant in equation (2) which is equal to 26 K_n. These data may be used as a guide during the field reconnaissance to establish an appropriate K_n value for the drainage basin being studied. As previously stated, it is of considerable value to conduct a field reconnaissance of the basins represented in the data set to gain an understanding of the physical conditions that are indicative of a particular K_n value.

Figure 4-6 and the data in table 4-1 represent conditions on the Great Plains west of the Mississippi River and east of the foothills of the Rocky Mountains. The relationships shown on figure 4-6 reflect K_n values from 0.069 to as low as 0.030, which result in lag equation coefficients C_t of 1.8 and 0.77, respectively. The upper limit values generally reflect basins with considerable overland flow before reaching moderately well-defined watercourses. Many upper reach watercourses are swales, and the well-defined drainage networks are limited to the lower parts of the basins. Overbank flow conditions reflect relatively high Manning's n values. The lower limit values generally reflect a well-defined drainage network reaching points near the basin boundary, the overland flow occurs for fairly short distances before entering a well-defined watercourse, and the overbank conditions reflect relatively low Manning's n values.

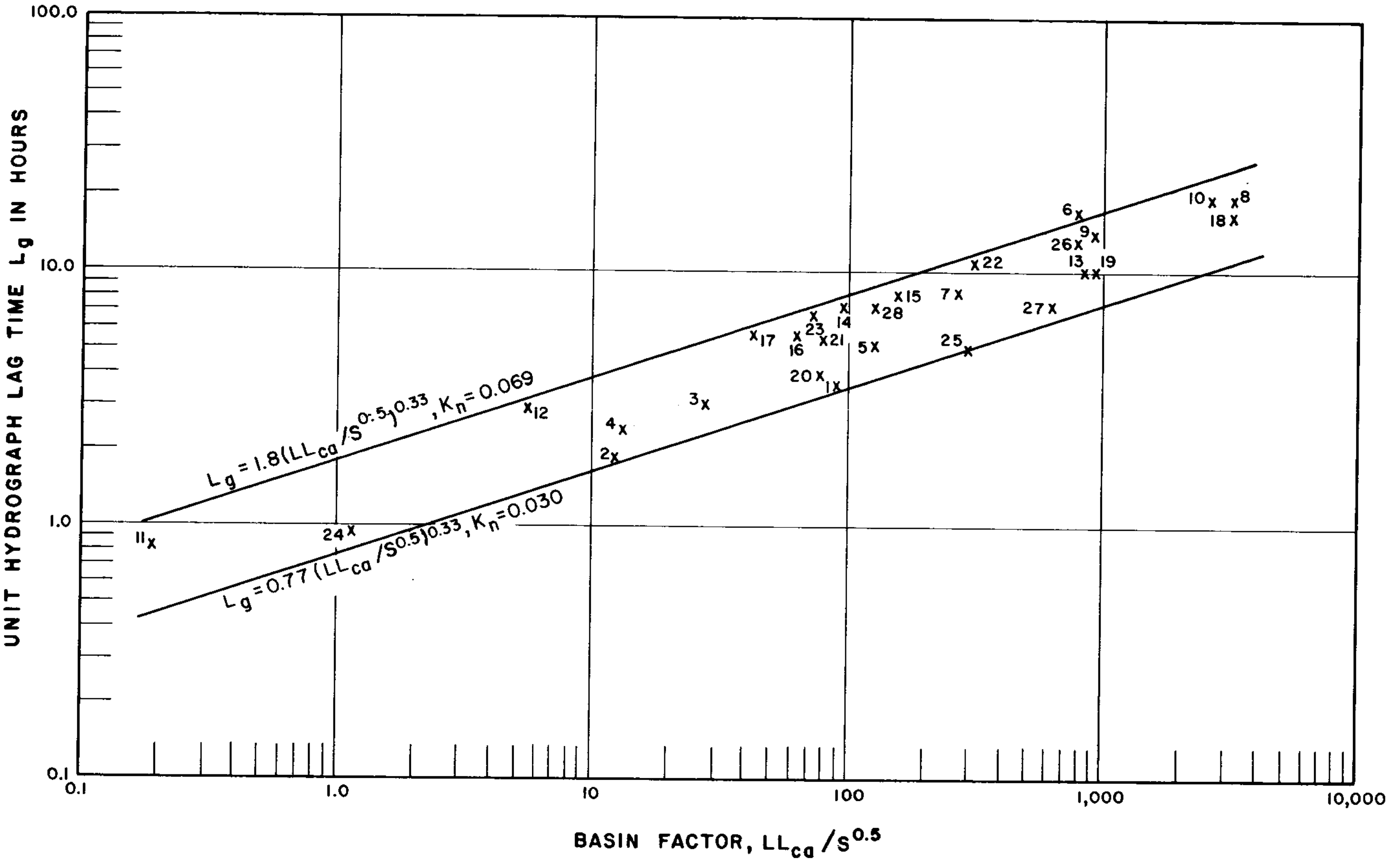


Figure 4-6.—Unit hydrograph lag relationships for the Great Plains. 103-D-1850.

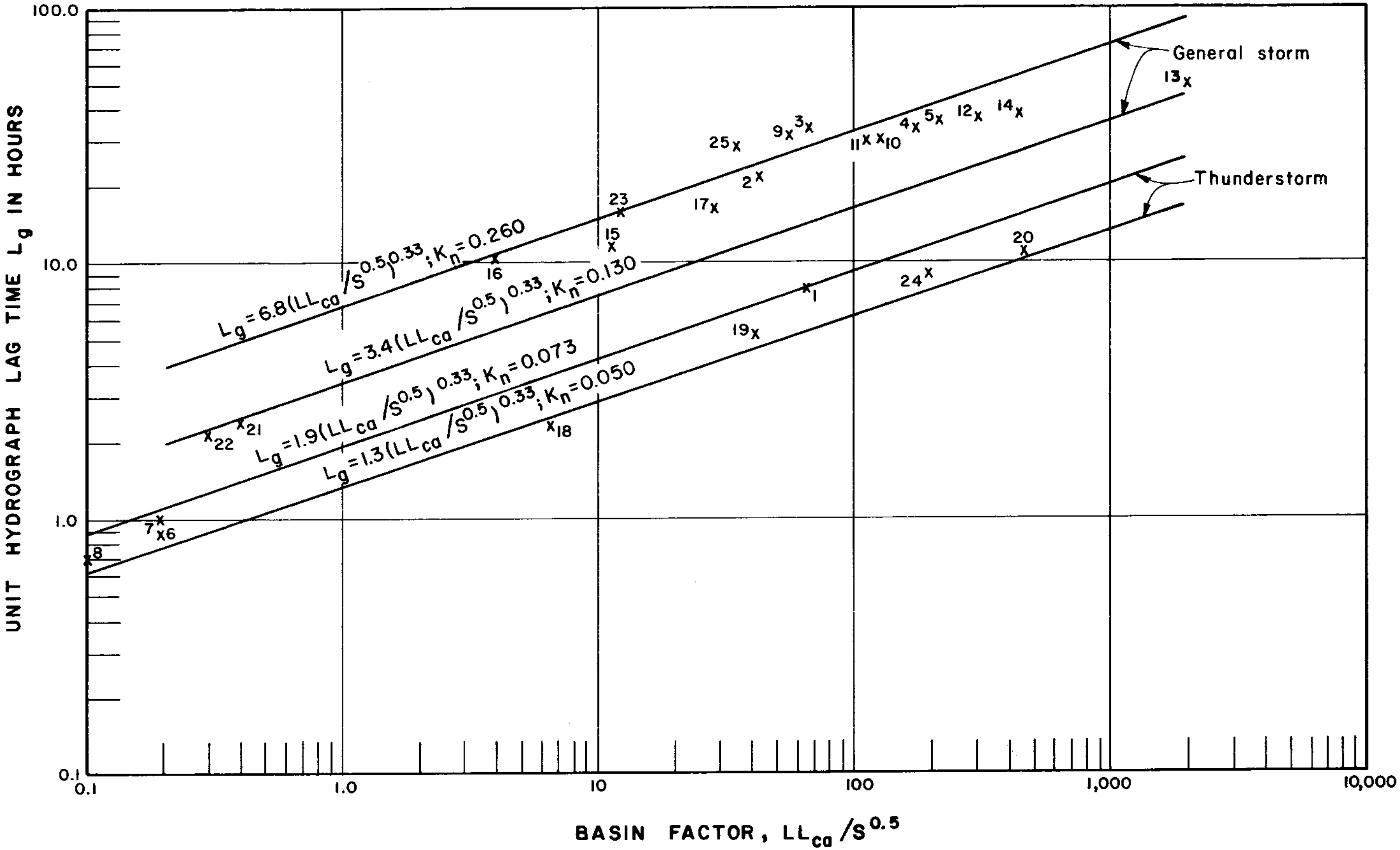


Figure 4-7.—Unit hydrograph lag relationships for the Rocky Mountains. 103-D-1851.

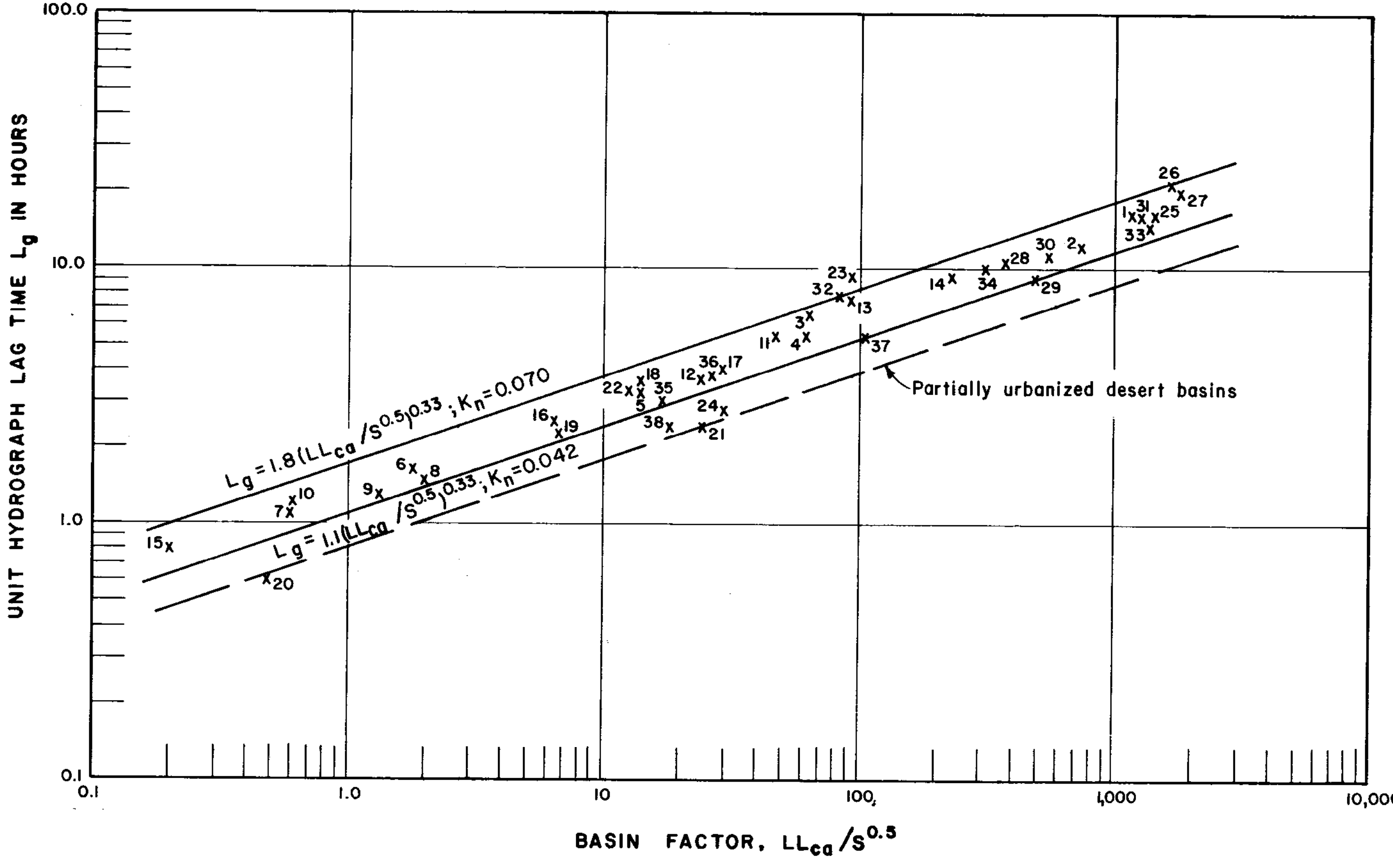


Figure 4-8.—Unit hydrograph lag relationships for the Southwest Desert, Great Basin, and Colorado Plateau. 103-D-1852.

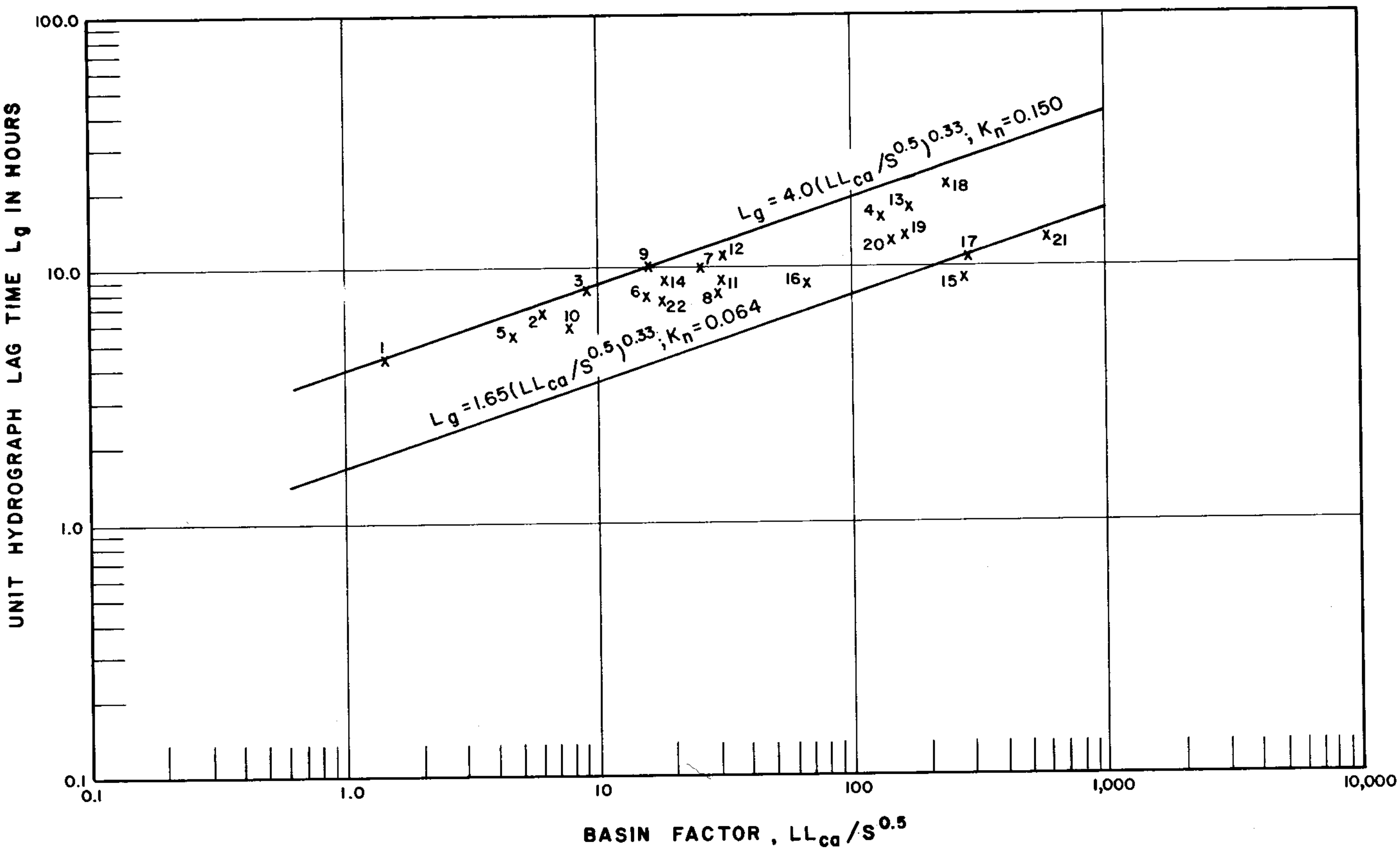


Figure 4-9.—Unit hydrograph lag relationships for the Sierra Nevada in California. 103-D-1853.

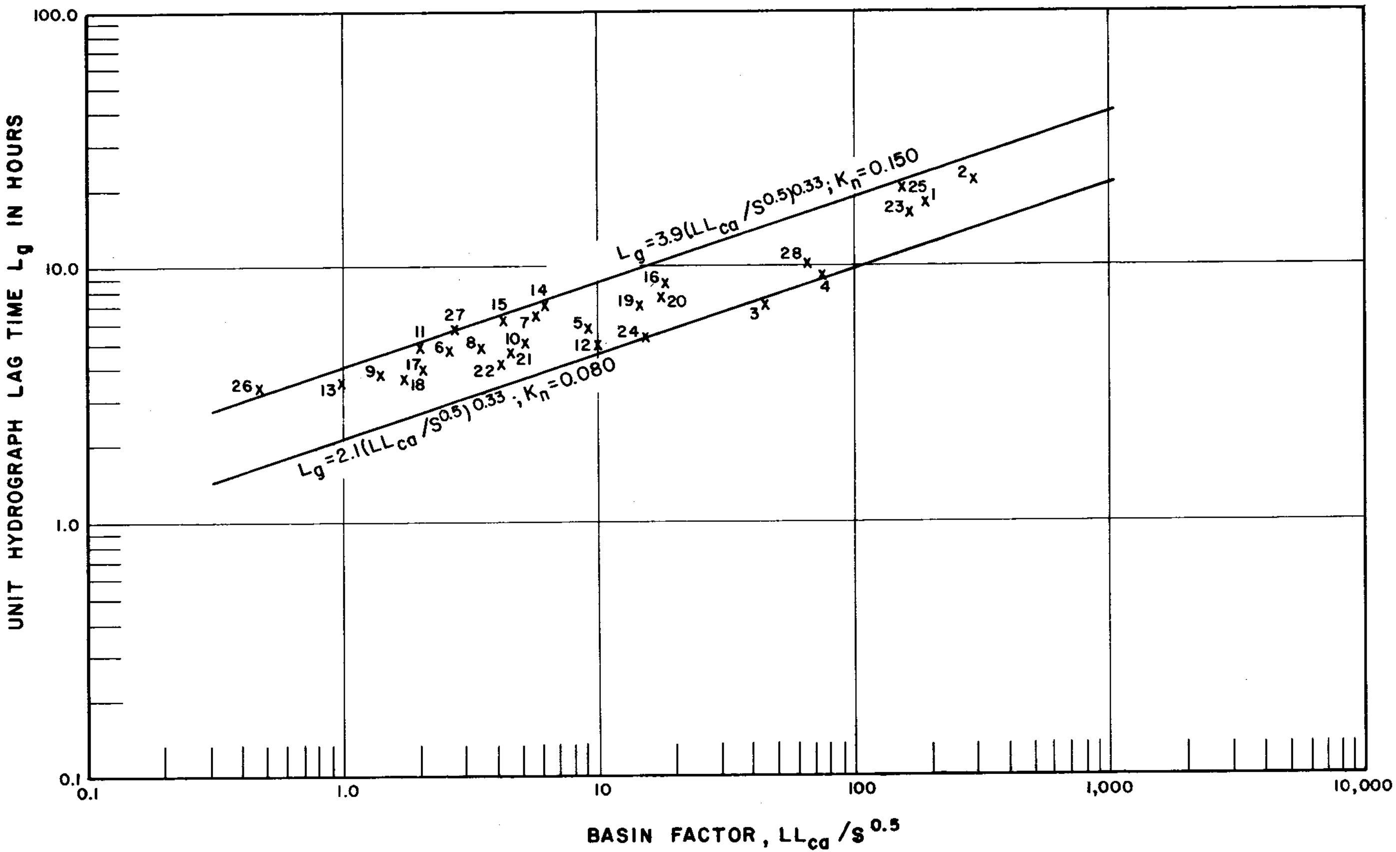


Figure 4-10.—Unit hydrograph lag relationships for the Coast and Cascade ranges of California, Oregon, and Washington. 103-D-1854.

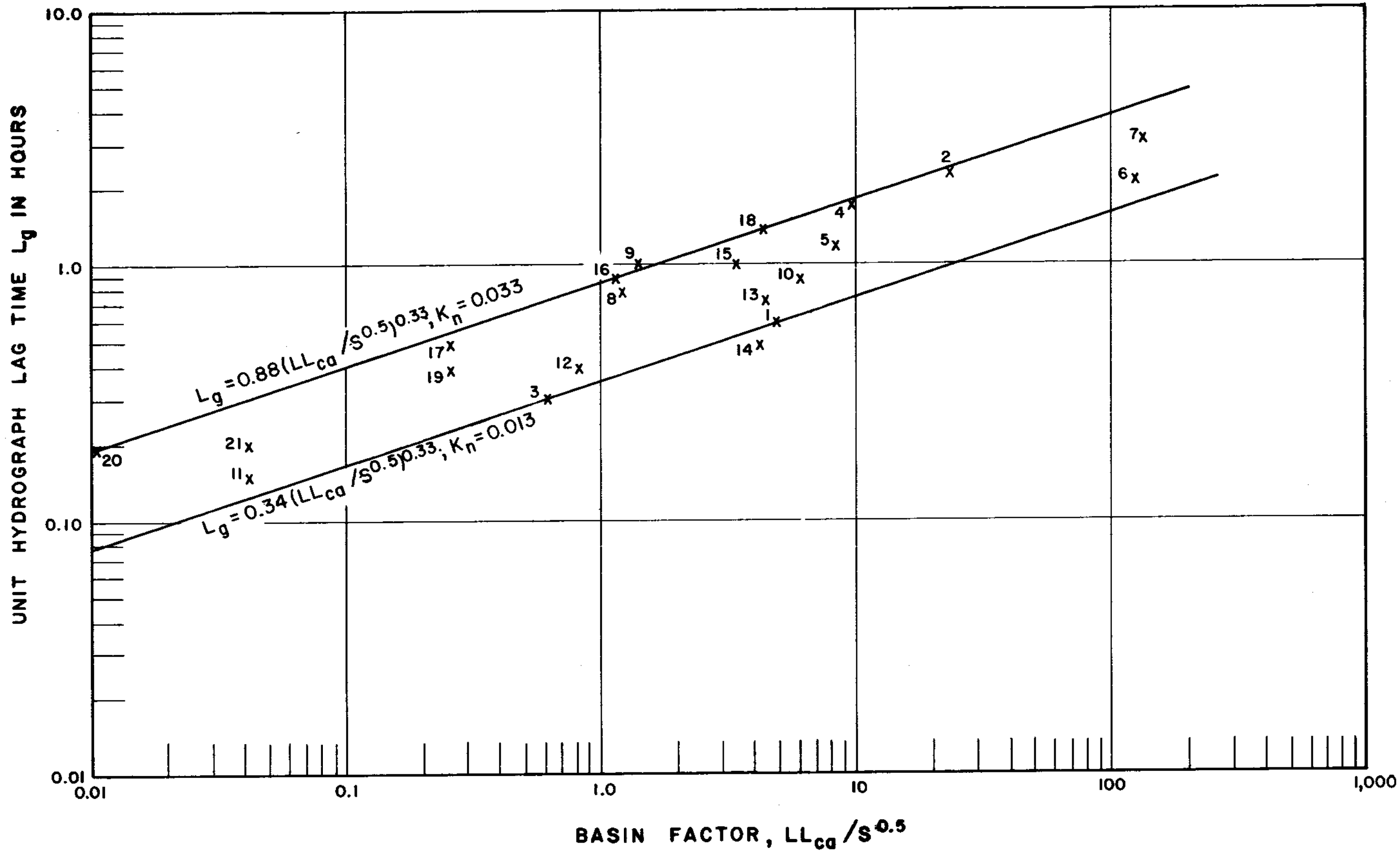


Figure 4-11.—Unit hydrograph lag relationships for urban basins. 103-D-1855.

Table 4-1.—Unit hydrograph lag data for the Great Plains.

Index No.	Station and location	Drainage area, mi²	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Black Squirrel Cr. nr. Ellicot, CO	353.0	92.9	3.5	0.030	0.78
2	Jimmy Camp Cr. nr. Widefield, CO	54.3	12.2	1.8	.030	0.78
3	Dry Creek nr. Lamar, CO	73.0	27.9	3.1	.040	1.04
4	Willow Cr. nr. Lamar, CO	40.5	13.3	2.5	.041	1.07
5	Clay Cr. above Clay Cr. Dam nr. Lamar, CO	213.0	129.0	5.2	.040	1.04
6	Smokey Hill R. nr. Ellsworth, KS	[1]1050.0	787.0	17.9	.076	1.98
7	Cimmaron R. nr. Boise City, OK	2150.0	275.0	8.4	.051	1.33
8	North Fk. Red R. nr. Granite, OK	[1]2005.0	3230.0	20.0	.053	1.38
9	Elm Fk. of North Fk. Red R. nr. Magnum, OK	838.0	920.0	14.5	.076	1.98
10	Salt Fk. Red R. nr. Magnum, OK	1566.0	2645.0	21.0	.060	1.56
11	Beaver Cr. No. 3 NE (Central Plains Experiment Station)	2.0	0.19	0.88	.059	1.53
12	Beaver Cr. No. 8, NE (Central Plains Experiment Station)	25.0	5.7	3.1	.067	1.74
13	Washita R. at Clinton, OK	794.0	860.0	10.5	.043	1.12
14	Barnitze Cr. nr. Arapaho, OK	243.0	99.0	7.5	.063	1.64
15	Pond Cr. nr. Ft. Cobb, OK	300.0	156.0	8.4	.061	1.59
16	Rock Cr. nr. Dougherty, OK	134.0	65.9	5.8	.056	1.46
17	Red Willow Cr. nr. McCook, NE	[1]68.0	44.4	5.8	.064	1.66
18	Pecos R. at Puerto D. Lune, NM	3970.0	3300.0	17.0	.045	1.17
19	Pecos R. at Anton Chico, NM	1050.0	890.0	10.5	.043	1.12
20	Vermejo R. at Dawson, NM	299.0	83.0	4.2	.038	0.99
21	Vermejo R. at Dawson, NM (2d reconstruction)	299.0	83.0	5.7	.051	1.33
22	Rio Hondo nr. Diamond A Ranch, NM	960.0	312.0	11.0	.064	1.66
23	Rio Ruidoso nr. Hondo, NM	307.0	73.5	7.0	.065	1.69
24	Buckhorn Cr. nr. Masonville, CO	[1]6.9	1.2	1.0	.036	0.94
25	Washita R. nr. Cheyenne, OK	353.0	306.0	5.1	.030	0.78
26	Medicine Cr. nr. Cambridge, NE	722.0	797.0	13.5	.057	1.48
27	Little Beaver Cr. above Marmath, ND	550.0	648.0	7.7	.035	0.91
28	Middle Fk. Powder R. above Kaycee, WY	980.0	131.0	7.7	.059	1.53

[1]Contributing area

Figure 4-7 and the data in table 4-2 represent conditions in the Rocky Mountains. Included are the Front, Sangre de Christo, San Juan, Wasatch, Big Horn, Absaroka, Wind River, and Bitteroot Ranges of New Mexico, Colorado, Wyoming, Utah, Idaho, Oregon, and Montana. Unit hydrograph lag time data representing basins located at the higher elevations of these mountain ranges are generally lacking. In addition, the infrequency of severe rainstorms in these areas and in the Northern States precludes acquisition of a good data base representing severe event phenomena. Examination of the available data has led to the conclusion that they represent two types of storm phenomena, the low-intensity general storm and the high-intensity thunderstorm event. Accordingly, two sets of relationships are presented on figure 4-7, one for each type meteorologic event.

Relationships shown on figure 4-7, representing the general storm phenomena, indicate K_n values of a high of 0.260 to a low of 0.130 with resulting lag equation coefficient values of 6.8 and 3.4, respectively. Because most of the data reflect low-intensity storms, a K_n of 0.160 (lag equation coefficient C_t of 4.2) or less should be used in the development of PMF hydrographs. This limiting value is consistent with data for drainage basins located in the Sierra Nevada of California, which have hydrologic characteristics similar to those of the Rocky Mountains. Higher values are considered appropriate for developing flood hydrographs of more common frequency than the 100-year event, for example.

Relationships shown on figure 4-7, representing the thunderstorm phenomena in this region, indicate lag equation coefficient C_t values ranging from 1.9 and 1.3 for K_n values ranging from 0.073 to 0.050, respectively. Selection of a value within these limits depends primarily on the character of flow retarding vegetation in the portions of the basin where overland flow will occur in the overbank flow areas, on the bed material in the principal channels, and also on the extent to which the drainage network has been developed by erosion.

Figure 4-8 and the data in table 4-3 represent conditions in the Southwest Desert, Great Basin, and Colorado Plateau regions of southern California, Nevada, Utah, Arizona, and western Colorado and New Mexico. Basins in this arid region generally have sparse vegetation, fairly well-defined drainage networks, and terrain varying from rolling to very rugged in the more mountainous areas.

Relationships shown on figure 4-8, reflecting relatively high hydraulic efficiencies, indicate regional K_n values range from a high of 0.070 to a low of 0.042 with lag equation coefficient C_t values of 1.8 to 1.1, respectively. The higher value is indicative of decreased basin hydraulic efficiency consistent with the coniferous forests found at the higher elevations in this desert region, and the lower value is typical of the usual desert terrain. The third lag curve, dashed line on figure 4-8, represents

Table 4-2.—Unit hydrograph lag data for the Rocky Mountains, New Mexico, Colorado, Utah, Wyoming, Montana, Idaho, and Oregon.

Index No.	Station and location	Drainage area, mi²	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Purgatoire R. at Trinidad, CO	742.0	69.8	8.0	0.076	1.98
2	Wood R. nr. Meeteetse, WY	194.0	41.9	21.5	.241	6.27
3	Grey Bull R. nr. Meeteetse, WY	681.0	68.3	34.0	.324	8.42
4	San Miguel R. at Naturita, CO	1080.0	174.0	34.0	.238	6.19
5	Uncompaghre R. at Delta, CO	1110.0	216.0	36.0	.235	6.11
6	Dry Gulch nr. Estes Park, CO	2.1	0.2	0.9	.059	1.53
7	Rabbit Gulch nr. Estes Park, CO	3.4	0.2	1.0	.065	1.69
8	North Fk. Big Thompson R. nr. Glen Haven, CO	1.3	0.1	0.7	.058	1.51
9	Uintah R. nr. Neola, UT	181.0	59.0	32.0	.324	8.42
10	South Fk. Payette R. nr. Garden Valley, ID	779.0	123.0	30.0	.236	6.14
11	Malheur R. nr. Drewsey, OR	910.0	114.0	30.0	.242	6.29
12	Weiser R. above Craney Cr. nr. Weiser, ID	1160.0	310.0	37.0	.214	5.56
13	Madison R. nr. Three Forks, MT	2511.0	2060.0	50.0	.155	4.03
14	Gallatin R. at Logan, MT	1795.0	443.0	38.0	.196	5.10
15	Surface Cr. at Cedaredge, CO	43.0	11.3	11.3	.195	5.07
16	South Piney Cr. at Willow Park, WY	28.9	3.8	10.5	.260	6.76
17	Piney Cr. at Kearney, WY	106.0	29.0	16.5	.209	5.43
18	Coal Cr. nr. Cedar City, UT	92.0	6.6	2.4	.050	1.30
19	Sevier R. nr. Hatch, UT	260.0	41.0	5.1	.058	1.51
20	Sevier R. nr. Kingston, UT	1110.0	469.0	11.0	.056	1.46
21	Centerville Cr. nr. Centerville, UT	3.9	0.4	2.4	.124	3.22
22	Parrish Cr. nr. Centerville, UT	2.0	0.3	2.2	.126	3.28
23	Florida R. nr. Hermosa, CO	69.4	12.5	15.5	.259	6.73
24	Dolores R. nr. McPhee, CO	793.0	193.0	9.0	.061	1.59
25	Los Pinos R. nr. Bayfield, CO	284.0	35.0	28.5	.339	8.81

Table 4-3.—Unit hydrograph lag data for the Southwest Desert, Great Basin, and Colorado Plateau.

Index No.	Station and location	Drainage area, mi^2	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Salt River at Roosevelt, AZ	4341.0	1261.0	16.0	0.058	1.51
2	Verde R. above E. Verde and below Jerome, AZ	3190.0	760.0	12.0	.052	1.35
3	Tonto Cr. above Gun Cr., AZ	678.0	66.3	6.5	.063	1.64
4	Agua Fria R. nr. Mayor, AZ	590.0	63.2	5.4	.053	1.38
5	San Gabriel R. at San Gabriel Dam, CA	162.0	14.4	3.3	.053	1.38
6	West Fk. San Gabriel R. at Cogswell Dam, CA	40.4	1.8	1.6	.051	1.33
7	Santa Anita Cr. at Santa Anita Dam, CA	10.8	0.6	1.1	.050	1.30
8	Sand Dimas Cr. at San Dimas Dam, CO	16.2	2.0	1.5	.046	1.20
9	Eaton Wash at Eaton Wash Dam, CA	9.5	1.3	1.3	.046	1.20
10	San Antonio Cr. nr. Claremont, CA	16.9	0.6	1.2	.055	1.43
11	Santa Clara R. nr. Saugus, CA	355.0	48.2	5.6	.060	1.56
12	Temecula Cr. at Pauba Canyon, CA	168.0	24.1	3.7	.050	1.30
13	Santa Margarita R. nr. Fallbrook, CA	645.0	99.2	7.3	.062	1.61
14	Santa Margarita R. at Ysidora, CA	740.0	228.0	9.5	.061	1.59
15	Live Oak Cr. at Live Oak Dam, CA	2.3	0.2	0.8	.052	1.35
16	Tujunga Cr. at Big Tujunga Dam, CA	81.4	6.5	2.5	.052	1.35
17	Murrieta Cr. at Temecula, CA	220.0	28.9	4.0	.051	1.33
18	Los Angeles R. at Sepulveda Dam, CA	152.0	14.3	3.5	.056	1.46
19	Pacoima Wash at Pacoima Dam, CA	27.8	6.8	2.4	.049	1.27
20	East Fullerton Cr. at Fullerton Dam, CA	3.1	0.5	0.6	.029	0.75
21	San Jose Cr. at Workman Mill Rd. CA	81.3	24.8	2.4	.032	0.83
22	San Vincente Cr. at Foster, CA	75.0	12.8	3.2	.053	1.38
23	San Diego R. nr. Santee, CA	380.0	95.4	9.2	.078	2.03
24	Deep Cr. nr. Hesperia, CA	137.0	28.1	2.8	.036	0.94
25	Bill Williams R. at Planet, AZ	4730.0	1476.0	16.2	.056	1.46
26	Gila R. at Conner No. 4 Damsite, AZ	2840.0	1722.0	21.5	.071	1.85
27	San Francisco R. at Jct. with Blue R., AZ	2000.0	1688.0	20.6	.068	1.77
28	Blue R., nr. Clifton, AZ	790.0	352.0	10.3	.057	1.48
29	Moencopi Wash nr. Tuba City, AZ	2490.0	473.0	9.2	.046	1.20
30	Clear Cr. nr. Winslow, AZ	607.0	570.0	11.2	.053	1.38
31	Puerco R. nr. Admana, AZ	2760.0	1225.0	15.9	.058	1.51
32	Plateau Cr. nr. Cameo, CO	604.0	89.9	7.9	.069	1.79
33	White R. nr. Watson, UT	4020.0	1473.0	15.7	.054	1.40
34	Paria R. at Lees Ferry, AZ	1570.0	296.0	10.2	.060	1.56
35	New River at Rock Springs, AZ	67.3	16.5	3.1	.047	1.22
36	New River at New River, AZ	85.7	26.3	3.7	.048	1.25
37	New R. at Bell Road nr. Phoenix, AZ	187.0	108.0	5.3	.043	1.12
38	Skunk Cr. nr. Phoenix, AZ	64.6	18.7	2.4	.035	0.91

partially urbanized basins in the desert region. The position of this curve, below the two limiting curves, reflects the increased hydraulic efficiency normally found to be associated with the urbanization of a drainage basin.

Figure 4-9 and data in table 4-4 represent conditions in the Sierra Nevada of California. Basins in this region normally have well-developed drainage networks and substantial coniferous growth throughout those parts of the basins above about elevation 2000. River and stream channels are generally well incised into the bedrock. The hydrologic and hydraulic characteristics of the Sierra Nevada basins are mostly quite similar to those of the Rocky Mountains. However, the greater amount of data available for the Sierra Nevada reflect flood hydrograph reconstitutions for flood events resulting from major, intense storms. Such is not the case for the Rockies, which is the reason for establishing the upper limit K_n at 0.160, or a lag equation coefficient of 0.42, used for generating PMF's for basins in the Rocky Mountain region. Relationships shown on figure 4-9, reflecting the varying degrees of hydraulic efficiency, show that K_n ranges from a high of 0.150 to a low of 0.064 with associated lag equation coefficients of 4.0 to 1.65, respectively. However, considering the few points shown on figure 4-9 at or near the lower value, care should be taken before selecting a low K_n to ensure that the basin being studied has essentially the same hydraulic efficiency characteristics in terms of geology, drainage network development, and stream hydraulic roughness as those in the data set shown in table 4-4.

Again, it should be emphasized that the lag data accumulated for a condition may not reflect the hydraulic conditions present in a PMF event. Therefore, when attempting to assign values representative of those present during a PMF, a conservative approach should be taken and values assigned toward the low end of the scale for those conditions cited in the previous paragraphs.

Figure 4-10 and the accompanying data tabulated in table 4-5 represent conditions in the Coast and Cascade Ranges of California, Oregon, and Washington. Relationships shown on figure 4-10 indicate at the high end of the K_n range, a value of 0.150, or a lag equation coefficient C_t of 3.9. This is indicative of very heavy coniferous growth extending into the overbank flood plain, which lowers the hydraulic efficiencies of these basins. At the low end of the K_n range, a value of 0.080 (C_t of 2.1) is typical of the low lying basins where considerably sparser vegetation results in a higher hydraulic efficiency.

Figure 4-11 and data in table 4-6 represent urban conditions at several locations throughout the United States. As shown by the relationships on figure 4-11, the range in K_n values, from 0.033 to 0.013, primarily reflects the density and type of development and extent to which engineered floodwater collection systems have been constructed. Associated lag equation coefficients range from 0.88 to 0.34, respectively. A high-density development combined with a good collection system is typical

Table 4-4.—Unit hydrograph lag data for the Sierra Nevada in California.

Index No.	Station and location	Drainage area, mi^2	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Pitman Cr. below Tamarack Cr., CA	22.7	1.4	4.4	0.151	3.93
2	North Fk. Kings R. nr. Cliff Camp, CA	[1]70.0	6.2	6.7	.141	3.67
3	North Fk. Kings R. below Rancheria, CA	[1]116.0	9.2	8.4	.155	4.03
4	Cosumnes R. at Michigan Bar, CA	537.0	133.0	16.0	.123	3.20
5	Cosgrove Cr. nr. Valley Springs, CA	20.6	4.6	5.5	.128	3.33
6	Woods Cr. nr. Jacksonville, CA	98.4	15.1	7.8	.122	3.17
7	North Fk. Calaveras R. nr. San Andreas, CA	85.7	25.4	10.0	.132	3.43
8	Calaveras R. at Calaveras Reservoir, CA	395.0	30.6	8.5	.106	2.76
9	Calaveritas Cr. nr. San Andreas, CA	53.3	15.6	10.0	.155	4.03
10	North Fk. Cosumnes R. at Cosumnes Mine, CA	36.9	7.7	6.0	.118	3.07
11	Tule R. at Success Dam, CA	388.0	31.4	8.8	.109	2.83
12	Kaweah R. at Terminus Dam, CA	560.0	30.4	11.5	.143	3.72
13	Kings R. at Pine Flat Dam, CA	1542.0	168.0	17.2	.122	3.17
14	Big Dry Cr. Reservoir, CA	86.0	18.5	9.2	.135	3.51
15	Stanislaus R. at Melones Dam, CA	897.0	269.0	9.2	.056	1.46
16	Calaveras R. at Hogan Reservoir, CA	363.0	66.0	8.6	.083	2.16
17	American R. at Folsom Dam, CA	1875.0	290.0	10.9	.065	1.69
18	Kern R. at Isabella Dam, CA	2075.0	235.0	21.5	.136	3.54
19	North Yuba R. at Bullard's Bar Dam, CA	481.0	164.0	13.2	.094	2.44
20	Yuba R. at Englebright Dam, CA	990.0	143.0	12.5	.093	2.42
21	San Joaquin R. at Friant Dam, CA	1261.0	497.0	13.7	.068	1.77
22	South Fk. Consumnes R. nr. River Pines, CA	64.3	17.7	7.6	.113	2.94

[1]Contributing area.

Table 4-5.—Unit hydrograph lag data for the Coast and Cascade ranges in California, Oregon, and Washington.

Index No.	Station and location	Drainage area, mi²	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Putah Cr. nr. Winters, CA	577.0	190.0	17.5	0.119	3.09
2	Stony Cr. nr. Hamilton City, CA	764.0	288.0	21.8	.129	3.35
3	Huasna R. nr. Santa Maria, CA	119.0	45.4	7.0	.076	1.98
4	Sisquoc R. nr. Garey, CA	465.0	76.8	8.9	.082	2.13
5	Salinas R. nr. Pozo, CA	114.0	9.0	5.7	.106	2.76
6	Corte Madera Cr. at Ross, CA	18.1	2.6	4.6	.129	3.35
7	East Fk. Russian R. nr. Calpella, CA	93.0	5.9	6.5	.139	3.61
8	Novato Cr. nr. Novato, CA	17.5	3.5	4.7	.120	3.12
9	Pinole Cr. nr. Pinole, CA	10.0	1.4	3.8	.131	3.41
10	San Francisquito Cr. nr. Stanford University, CA	38.3	4.8	4.8	.110	2.86
11	San Lorenzo Cr. at Hayward, CA	37.5	2.0	4.9	.150	3.90
12	Sonoma Cr. at Boyes Hot Springs, CA	62.2	10.0	4.8	.086	2.24
13	Corralitos Cr. nr. Corralitos, CA	10.6	0.97	3.4	.132	3.43
14	Austin Cr. nr. Cadzadero, CA	63.0	6.2	6.8	.143	3.72
15	Dry Cr. nr. Napa, CA	17.4	4.3	6.0	.143	3.72
16	South Fk. Eel R. nr. Branscomb, CA	43.9	17.8	8.1	.120	3.12
17	Branciforte Cr. at Santa Cruz, CA	17.3	2.1	3.9	.117	3.04
18	Matadero Cr. at Palo Alto, CA	7.2	1.7	3.7	.119	3.09
19	Napa R. at St. Helena, CA	81.1	14.8	6.8	.107	2.78
20	San Lorenzo R. at Big Trees, CA	111.0	17.8	8.0	.119	3.09
21	Uvas Cr. at Morgan Hill, CA	30.4	4.4	4.4	.104	2.70
22	Feliz Cr. nr. Hopland, CA	31.2	4.0	3.9	.095	2.47
23	Redwood Cr. at Orick, CA	278.0	170.0	16.0	.113	2.94
24	Russian R. at Ukiah, CA	99.6	14.5	5.1	.081	2.11
25	Trinity R. at Lewiston, CA	726.0	157.0	20.0	.145	3.77
26	Powell Cr. nr. Williams, OR	8.6	0.47	3.4	.168	4.37
27	Slate Cr. nr. Wonder, OR	30.9	2.8	5.6	.153	3.98
28	Arroyo Del Valle nr. Livermore, CA	147.0	66.5	10.0	.096	2.50

Table 4-6.—Unit hydrograph lag data for urban basins.

Index No.	Station and location	Drainage area, mi²	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Alhambra Wash above Short St., Monterey Park, CA	14.0	4.8	0.6	0.011	0.29
2	San Jose Cr. at Workman Mill Rd., Whittier, CA	81.3	24.8	2.4	.032	0.83
3	Broadway Drain at Raymond Dike, CA	2.5	0.6	0.3	.014	0.36
4	Compton Cr. below Hooper Ave. Storm Drain, L.A., CA	19.5	9.7	1.8	.033	0.86
5	Ballona Cr. at Sawtelle Blvd., L.A., CA	88.6	8.3	1.2	.023	0.60
6	Brays Bayou, Houston, TX	88.4	121.0	2.1	.017	0.44
7	White Oak Bayou, Houston, TX	92.0	134.0	3.1	.024	0.62
8	Boneyard Cr., Austin, TX	4.5	1.2	0.8	.029	0.75
9	Waller Cr., Austin, TX	4.1	1.4	1.0	.034	0.88
10	Beargrass Cr., Louisville, KY	9.7	5.6	0.9	.020	0.52
11	17th Street Sewer, Louisville, KY	0.2	0.04	0.15	.017	0.44
12	Northwest Trunk, Louisville, KY	1.9	0.8	0.4	.014	0.36
13	Southern Outfall, Louisville, KY	6.4	4.4	0.7	.017	0.44
14	Southwest Outfall, Louisville, KY	7.5	4.1	0.50	.012	0.31
15	Beargrass Cr., Louisville, KY	6.3	3.4	1.0	.026	0.68
16	Tripps Run nr. Falls Church, VA	4.6	1.1	0.9	.033	0.86
17	Tripps Run at Falls Church, VA	1.8	0.26	0.5	.030	0.78
18	Four Mile Run at Alexandria, VA	14.4	4.2	1.4	.034	0.88
19	Little Pimmit Run at Arlington, VA	2.3	0.25	0.4	.024	0.62
20	Piney Branch at Vienna, VA	0.3	0.01	0.2	.035	0.91
21	Walker Avenue Drain at Baltimore, MD	0.2	0.04	0.2	.022	0.57

of fully urbanized drainage basins with the lower K_n values. Low-density or partial development with only minor floodwater collection facilities are typical of basins with the higher K_n values. As a result, it is imperative that anticipated future developments be considered. Most urban development eventually tends to become high-density and, with continued flooding problems, also tends to have more extensive storm water collection systems. The hydrologic engineer must anticipate such eventualities and assign lower K_n values that could reasonably be expected with full future development during the functional life of the project being studied.

(f) Dimensionless Unit Hydrograph or S-Graph Selection.—Data gathered during the field reconnaissance along with a careful inspection of the drainage basin features as outlined and presented on the topographic map are compared with similar data for basins where unit hydrographs have been developed from analysis of observed flood hydrographs. The dimensionless unit hydrograph or S-graph selected should represent a drainage basin that is as similar to the ungauged basin as possible with respect to basin shape, topography, vegetal cover, drainage network development, and channel hydraulic characteristics. The most appropriate dimensionless unit hydrograph or S-graph to be used for a flood study would be one developed from analysis of a flood occurring in the basin under study. However, this is usually impossible because the records, if any, do not include a flood suitable for reconstitution, or no flood event may have occurred. Therefore, the analyst must resort to relationships for nearby hydrologically homogeneous basins. Currently, the Bureau's Denver Office has a catalog of over 30 of these relationships representing conditions for basins throughout the 17 Western States, and each regional office has copies of those that are applicable to drainage basins within their regional boundaries. Examination of these relationships has led to the conclusion that the typical ones shown in tables 4-7 through 4-18, for each of the five geographic regions and for the urbanized conditions, will produce satisfactory results in unit hydrograph development. However, the hydrologic engineer can improve results if sufficient knowledge of the drainage basin being studied is gained through field reconnaissances so that the most appropriate selection is made.

(g) Computing Synthetic Unit Hydrograph Ordinates.—Once the unit hydrograph lag time has been determined and the dimensionless unit hydrograph or S-graph has been selected, it is basically a mechanical process to determine the synthetic unit hydrograph ordinates. This process for each of the two techniques currently in use by the Bureau is discussed in the following two subsections.

(1) *Dimensionless unit hydrograph technique.*—The first item that must be determined is the unit duration of the synthetic unit hydrograph. To provide adequate definition near and at the peak of the unit hydrograph as well as the eventual flood hydrograph, the unit duration should approximate the lag time divided by 5.5 [55]. This calculated unit duration

Table 4-7.—Dimensionless unit hydrograph data for the Great Plains.

Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q
5	0.10	105	15.04	205	3.18	305	1.37	405	0.65	505	0.30
10	.20	110	13.52	210	2.98	310	1.32	410	.62	510	.29
15	.81	115	12.51	215	2.79	315	1.27	415	.60	515	.29
20	1.66	120	11.40	220	2.67	320	1.23	420	.58	520	.27
25	3.23	125	10.50	225	2.52	325	1.18	425	.56	525	.26
30	4.83	130	9.59	230	2.41	330	1.14	430	.54	530	.26
35	7.06	135	8.88	235	2.32	335	1.10	435	.52	535	.25
40	9.18	140	8.26	240	2.24	340	1.05	440	.50	540	.24
45	11.10	145	7.57	245	2.15	345	1.02	445	.48	545	.24
50	14.03	150	6.96	250	2.08	350	0.98	450	.46	550	.23
55	16.25	155	6.36	255	2.00	355	.94	455	.44	555	.22
60	18.07	160	5.95	260	1.92	360	.91	460	.43	560	.21
65	20.19	165	5.45	265	1.85	365	.87	465	.41	565	.20
70	21.40	170	5.05	270	1.79	370	.84	470	.40	570	.20
75	22.91	175	4.64	275	1.72	375	.81	475	.38	575	.19
80	24.02	180	4.39	280	1.66	380	.78	480	.37	580	.18
85	22.81	185	4.04	285	1.59	385	.75	485	.35	585	.18
90	20.59	190	3.78	290	1.54	390	.72	490	.34	590	.17
95	18.37	195	3.53	295	1.48	395	.70	495	.33	595	.16
100	16.65	200	3.38	300	1.42	400	.67	500	.32	600	.16

Table 4-8.—Dimensionless S-graph data for the Great Plains.

Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate
5	0.02	105	53.28	205	83.76	305	92.60	405	96.81	505	98.85
10	.06	110	56.25	210	84.42	310	92.89	410	96.95	510	98.93
15	.21	115	58.94	215	85.05	315	93.17	415	97.08	515	99.00
20	.52	120	61.43	220	85.63	320	93.44	420	97.21	520	99.08
25	1.11	125	63.71	225	86.19	325	93.70	425	97.34	525	99.15
30	2.01	130	65.81	230	86.72	330	93.95	430	97.46	530	99.22
35	3.31	135	67.74	235	87.22	335	94.19	435	97.58	535	99.29
40	5.02	140	69.53	240	87.70	340	94.43	440	97.69	540	99.35
45	7.11	145	71.20	245	88.16	345	94.65	445	97.80	545	99.41
50	9.70	150	72.73	250	88.61	350	94.87	450	97.91	550	99.48
55	12.76	155	74.15	255	89.04	355	95.08	455	98.01	555	99.53
60	16.20	160	75.46	260	89.46	360	95.28	460	98.11	560	99.59
65	20.02	165	76.67	265	89.86	365	95.48	465	98.20	565	99.65
70	24.17	170	77.80	270	90.25	370	95.66	470	98.29	570	99.70
75	28.57	175	78.84	275	90.62	375	95.85	475	98.38	575	99.76
80	33.23	180	79.80	280	90.98	380	96.02	480	98.47	580	99.81
85	37.95	185	80.70	285	91.33	385	96.19	485	98.55	585	99.86
90	42.39	190	81.54	290	91.66	390	96.35	490	98.63	590	99.91
95	46.40	195	82.33	295	91.99	395	96.51	495	98.70	595	99.95
100	50.00	200	83.07	300	92.30	400	96.66	500	98.78	600	100.00

Table 4-9.—General storm dimensionless unit hydrograph data for the Rocky Mountains.

Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q
5	0.26	105	11.91	205	3.72	305	1.63	405	0.74	505	0.34
10	.90	110	11.21	210	3.55	310	1.57	410	.71	510	.33
15	2.00	115	10.61	215	3.40	315	1.50	415	.68	515	.32
20	3.00	120	10.01	220	3.25	320	1.45	420	.65	520	.31
25	5.00	125	9.40	225	3.10	325	1.39	425	.63	525	.29
30	6.00	130	8.80	230	3.00	330	1.34	430	.60	530	.28
35	7.70	135	8.25	235	2.87	335	1.28	435	.56	535	.27
40	9.00	140	7.70	240	2.75	340	1.23	440	.58	540	.26
45	14.51	145	7.25	245	2.65	345	1.19	445	.54	545	.25
50	18.11	150	6.80	250	2.52	350	1.13	450	.52	550	.24
55	21.51	155	6.40	255	2.42	355	1.09	455	.50	555	.23
60	24.01	160	6.00	260	2.33	360	1.05	460	.48	560	.23
65	22.81	165	5.65	265	2.24	365	1.01	465	.46	565	.22
70	21.21	170	5.35	270	2.15	370	0.97	470	.44	570	.21
75	19.31	175	5.00	275	2.07	375	.93	475	.42	575	.20
80	16.91	180	4.80	280	1.99	380	.90	480	.41	580	.19
85	15.21	185	4.55	285	1.91	385	.86	485	.40	585	.19
90	14.21	190	4.30	290	1.83	390	.83	490	.38	590	.18
95	13.41	195	4.10	295	1.76	395	.80	495	.37	595	.17
100	12.71	200	3.90	300	1.70	400	.77	500	.35	600	.17

Table 4-10.—General storm dimensionless S-graph data for the Rocky Mountains.

Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate
5	0.05	105	52.51	205	81.06	305	91.68	405	96.55	505	98.82
10	.23	110	54.87	210	81.83	310	92.02	410	96.71	510	98.89
15	.62	115	57.10	215	82.56	315	92.35	415	96.86	515	98.96
20	1.20	120	59.21	220	83.26	320	92.67	420	97.01	520	99.04
25	2.15	125	61.20	225	83.93	325	92.97	425	97.15	525	99.11
30	3.46	130	63.08	230	84.57	330	93.26	430	97.29	530	99.19
35	4.97	135	64.84	235	85.18	335	93.55	435	97.42	535	99.26
40	6.72	140	66.50	240	85.78	340	93.82	440	97.54	540	99.33
45	9.33	145	68.05	245	86.35	345	94.08	445	97.66	545	99.39
50	12.74	150	69.51	250	86.89	350	94.33	450	97.78	550	99.46
55	16.84	155	70.88	255	87.42	355	94.58	455	97.89	555	99.52
60	21.47	160	72.17	260	87.92	360	94.81	460	98.00	560	99.58
65	26.17	165	73.39	265	88.41	365	95.03	465	98.11	565	99.64
70	30.58	170	74.53	270	88.87	370	95.25	470	98.21	570	99.69
75	34.66	175	75.62	275	89.32	375	95.45	475	98.31	575	99.75
80	38.32	180	76.64	280	89.75	380	95.65	480	98.40	580	99.80
85	41.57	185	77.61	285	90.17	385	95.85	485	98.49	585	99.85
90	44.55	190	78.54	290	90.57	390	96.03	490	98.58	590	99.90
95	47.35	195	79.43	295	90.95	395	96.21	495	98.66	595	99.95
100	50.00	200	80.26	300	91.32	400	96.38	500	98.74	600	100.00

Table 4-11.—Thunderstorm dimensionless unit hydrograph data for the Rocky Mountains.

Time *t*, in % of $L_g + 0.5\ D$	*q*	Time *t*, in % of $L_g + 0.5\ D$	*q*	Time *t*, in % of $L_g + 0.5\ D$	*q*	Time *t*, in % of $L_g + 0.5\ D$	*q*	Time *t*, in % of $L_g + 0.5\ D$	*q*	Time *t*, in % of $L_g + 0.5\ D$	*q*
5	0.14	105	20.76	205	2.75	305	1.05	405	0.43	505	0.18
10	.21	110	18.84	210	2.61	310	1.00	410	.42	510	.17
15	.33	115	16.81	215	2.44	315	0.96	415	.40	515	.17
20	.51	120	14.99	220	2.31	320	.92	420	.38	520	.16
25	.84	125	12.86	225	2.17	325	.88	425	.36	525	.16
30	1.62	130	11.04	230	2.04	330	.84	430	.35	530	.15
35	3.74	135	9.52	235	1.95	335	.81	435	.33	535	.15
40	6.38	140	8.41	240	1.84	340	.77	440	.32	540	.14
45	8.61	145	7.50	245	1.76	345	.74	445	.31	545	.14
50	10.94	150	6.69	250	1.69	350	.71	450	.29	550	.13
55	13.26	155	5.98	255	1.62	355	.68	455	.28	555	.13
60	15.70	160	5.47	260	1.55	360	.65	460	.27	560	.12
65	18.23	165	4.97	261	1.49	365	.62	465	.26	565	.12
70	20.76	170	4.55	270	1.42	370	.59	470	.25	570	.11
75	23.30	175	4.25	275	1.36	375	.57	475	.24	575	.11
80	25.83	180	3.89	280	1.30	380	.55	480	.23	580	.10
85	28.36	185	3.59	285	1.24	385	.52	485	.22	585	.10
90	26.53	190	3.34	290	1.19	390	.50	490	.21	590	.09
95	24.71	195	3.13	295	1.14	395	.48	495	.20	595	.09
100	22.68	200	2.93	300	1.09	400	.46	500	.19	600	.08

Table 4-12.—Thunderstorm dimensionless S-graph data for the Rocky Mountains.

Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate
5	0.03	105	54.43	205	87.87	305	95.09	405	98.11	505	99.40
10	.07	110	58.48	210	88.44	310	95.31	410	98.21	510	99.44
15	.14	115	62.14	215	88.97	315	95.52	415	98.30	515	99.48
20	.24	120	65.42	220	89.47	320	95.72	420	98.38	520	99.52
25	.40	125	68.32	225	89.95	325	95.92	425	98.46	525	99.55
30	.70	130	70.83	230	90.39	330	96.10	430	98.54	530	99.58
35	1.39	135	72.98	235	90.81	335	96.28	435	98.62	535	99.62
40	2.57	140	74.86	240	91.22	340	96.45	440	98.69	540	99.65
45	4.21	145	76.53	245	91.60	345	96.61	445	98.76	545	99.68
50	6.31	150	78.02	250	91.96	350	96.77	450	98.82	550	99.71
55	8.86	155	79.35	255	92.31	355	96.92	455	98.89	555	99.73
60	11.88	160	80.55	260	92.64	360	97.06	460	98.95	560	99.76
65	15.39	165	81.65	265	92.96	365	97.20	465	99.01	565	99.78
70	19.41	170	82.65	270	93.27	370	97.33	470	99.06	570	99.82
75	23.92	175	83.57	275	93.57	375	97.46	475	99.12	575	99.85
80	28.93	180	84.44	280	93.85	380	97.58	480	99.17	580	99.88
85	34.43	185	85.22	285	94.12	385	97.69	485	99.22	585	99.91
90	39.99	190	85.95	290	94.38	390	97.81	490	99.27	590	99.94
95	45.18	195	86.64	295	94.63	395	97.91	495	99.31	595	99.97
100	50.00	200	87.27	300	94.86	400	98.01	500	99.36	600	100.00

Table 4-13.—Dimensionless unit hydrograph data for the Southwest Desert, Great Basin, and Colorado Plateau.

Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q
5	0.19	105	18.92	205	3.47	305	1.15	405	0.38	505	0.12
10	.32	110	16.08	210	3.28	310	1.08	410	.36	510	.12
15	.48	115	14.19	215	3.10	315	1.02	415	.34	515	.11
20	.74	120	12.61	220	2.93	320	0.97	420	.33	520	.10
25	1.21	125	11.04	225	2.75	325	.91	425	.30		
30	1.81	130	9.99	230	2.63	330	.86	430	.28		
35	2.63	135	9.04	235	2.47	335	.82	435	.27		
40	3.68	140	8.20	240	2.33	340	.78	440	.26		
45	5.47	145	7.36	245	2.22	345	.74	445	.24		
50	8.41	150	6.78	250	2.10	350	.69	450	.23		
55	12.61	155	6.20	255	1.99	355	.66	455	.22		
60	16.50	160	5.83	260	1.88	360	.63	460	.21		
65	20.50	165	5.47	265	1.78	365	.59	465	.20		
70	23.97	170	5.15	270	1.68	370	.56	470	.19		
75	27.75	175	4.84	275	1.59	375	.53	475	.18		
80	28.91	180	4.57	280	1.50	380	.50	480	.17		
85	28.07	185	4.31	285	1.43	385	.47	485	.16		
90	26.38	190	4.10	290	1.36	390	.45	490	.15		
95	24.18	195	3.87	295	1.28	395	.42	495	.15		
100	21.55	200	3.68	300	1.21	400	.40	500	.13		

Table 4-14.—Dimensionless S-graph data for the Southwest Desert, Great Basin, and Colorado Plateau.

Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate
5	0.04	105	54.19	205	86.64	305	95.65	405	98.75	505	99.85
10	.10	110	57.86	210	87.36	310	95.89	410	98.84	510	99.89
15	.20	115	61.02	215	88.04	315	96.13	415	98.92	515	99.93
20	.34	120	63.83	220	88.68	320	96.35	420	98.99	520	99.97
25	.57	125	66.33	225	89.29	325	96.56	425	99.06	525	100.00
30	.91	130	68.53	230	89.86	330	96.75	430	99.13		
35	1.40	135	70.53	235	90.41	335	96.94	435	99.20		
40	2.08	140	72.34	240	90.93	340	97.12	440	99.26		
45	3.08	145	73.99	245	91.42	345	97.29	445	99.32		
50	4.57	150	75.47	250	91.88	350	97.45	450	99.37		
55	6.79	155	76.84	255	92.32	355	97.60	455	99.42		
60	9.79	160	78.10	260	92.74	360	97.74	460	99.47		
65	13.55	165	79.28	265	93.14	365	97.88	465	99.52		
70	18.03	170	80.40	270	93.51	370	98.01	470	99.57		
75	23.22	175	81.44	275	93.87	375	98.14	475	99.61		
80	28.90	180	82.43	280	94.21	380	98.25	480	99.65		
85	34.64	185	83.37	285	94.52	385	98.36	485	99.69		
90	40.15	190	84.25	290	94.83	390	98.47	490	99.73		
95	45.30	195	85.09	295	95.12	395	98.57	495	99.77		
100	50.00	200	85.88	300	95.39	400	98.66	500	99.81		

Table 4-15.—Dimensionless unit hydrograph data for the Sierra Nevada, Coast, and Cascade ranges.

Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q
5	0.65	105	13.83	205	3.89	305	1.92	405	1.00	505	0.43
10	1.30	110	12.53	210	3.73	310	1.85	410	0.96	510	.40
15	1.95	115	11.36	215	3.58	315	1.78	415	.93	515	.38
20	2.60	120	10.29	220	3.44	320	1.73	420	.90	520	.34
25	3.25	125	9.33	225	3.30	325	1.67	425	.87	525	.31
30	4.23	130	8.73	230	3.18	330	1.62	430	.84	530	.28
35	5.51	135	8.17	235	3.08	335	1.57	435	.82	535	.25
40	7.17	140	7.65	240	2.98	340	1.52	440	.80	540	.22
45	9.34	145	7.15	245	2.88	345	1.47	445	.77	545	.19
50	12.17	150	6.69	250	2.79	350	1.42	450	.75	550	.16
55	13.88	155	6.33	255	2.69	355	1.38	455	.72	555	.14
60	15.83	160	5.99	260	2.60	360	1.34	460	.69	560	.13
65	18.05	165	5.67	265	2.50	365	1.30	465	.66		
70	20.59	170	5.36	270	2.41	370	1.26	470	.63		
75	23.48	175	5.07	275	2.33	375	1.22	475	.61		
80	21.54	180	4.85	280	2.26	380	1.18	480	.58		
85	19.77	185	4.63	285	2.18	385	1.14	485	.55		
90	18.13	190	4.43	290	2.11	390	1.11	490	.52		
95	16.63	195	4.24	295	2.05	395	1.06	495	.49		
100	15.26	200	4.06	300	1.98	400	1.03	500	.46		

Table 4-16.—Dimensionless S-graph data for the Sierra Nevada, Coast, and Cascade ranges.

Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate	Time t, in % of L_g	Discharge, % of ultimate
5	0.14	105	52.79	205	80.49	305	91.47	405	97.00	505	99.66
10	.43	110	55.32	210	81.25	310	91.84	410	97.19	510	99.73
15	.86	115	57.60	215	81.98	315	92.20	415	97.38	515	99.79
20	1.44	120	59.66	220	82.68	320	92.55	420	97.56	520	99.84
25	2.17	125	61.57	225	83.35	325	92.89	425	97.73	525	99.89
30	3.13	130	63.35	230	84.00	330	93.22	430	97.90	530	99.92
35	4.38	135	65.01	235	84.63	335	93.53	435	98.06	535	99.96
40	6.04	140	66.56	240	85.24	340	93.83	440	98.22	540	99.99
45	8.21	145	68.01	245	85.83	345	94.13	445	98.36	545	100.00
50	10.94	150	69.38	250	86.40	350	94.41	450	98.51		
55	14.06	155	70.67	255	86.94	355	94.69	455	98.64		
60	17.64	160	71.89	260	87.47	360	94.96	460	98.78		
65	21.73	165	73.04	265	87.98	365	95.22	465	98.90		
70	26.42	170	74.13	270	88.47	370	95.47	470	99.02		
75	31.28	175	75.16	275	88.94	375	95.71	475	99.13		
80	35.72	180	76.15	280	89.40	380	95.94	480	99.23		
85	39.78	185	77.10	285	89.84	385	96.17	485	99.33		
90	43.50	190	78.00	290	90.27	390	96.39	490	99.42		
95	46.91	195	78.87	295	90.69	395	96.60	495	99.51		
100	50.00	200	79.70	300	91.08	400	96.81	500	99.59		

Table 4-17.—Dimensionless unit hydrograph data for urban basins.

Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q	Time t, in % of $L_g + 0.5\ D$	q
5	0.64	105	14.50	205	3.73	305	1.64	405	0.81	505	0.40
10	1.56	110	13.08	210	3.55	310	1.60	410	.78	510	.39
15	2.52	115	12.19	215	3.37	315	1.53	415	.75	515	.37
20	3.57	120	11.31	220	3.24	320	1.49	420	.73	520	.36
25	4.36	125	10.27	225	3.04	325	1.42	425	.69	525	.34
30	5.80	130	9.63	230	2.93	330	1.39	430	.67	530	.33
35	6.95	135	8.96	235	2.75	335	1.32	435	.64	535	.32
40	8.38	140	8.27	240	2.67	340	1.28	440	.62	540	.31
45	9.87	145	7.75	245	2.53	345	1.23	445	.60	545	.30
50	11.52	150	7.22	250	2.47	350	1.21	450	.58	550	.29
55	13.19	155	6.75	255	2.37	355	1.15	455	.56	555	.28
60	15.18	160	6.27	260	2.30	360	1.11	460	.54	560	.27
65	17.32	165	5.94	265	2.21	365	1.07	465	.52	565	.26
70	19.27	170	5.55	270	2.12	370	1.03	470	.50	570	.25
75	19.74	175	5.24	275	2.04	375	1.00	475	.49	575	.24
80	20.00	180	4.92	280	1.98	380	0.97	480	.48	580	.24
85	19.74	185	4.63	285	1.90	385	.93	485	.46	585	.23
90	19.27	190	4.39	290	1.83	390	.90	490	.45	590	.22
95	17.72	195	4.18	295	1.78	395	.87	495	.43	595	.21
100	16.12	200	3.93	300	1.71	400	.84	500	.41	600	.21

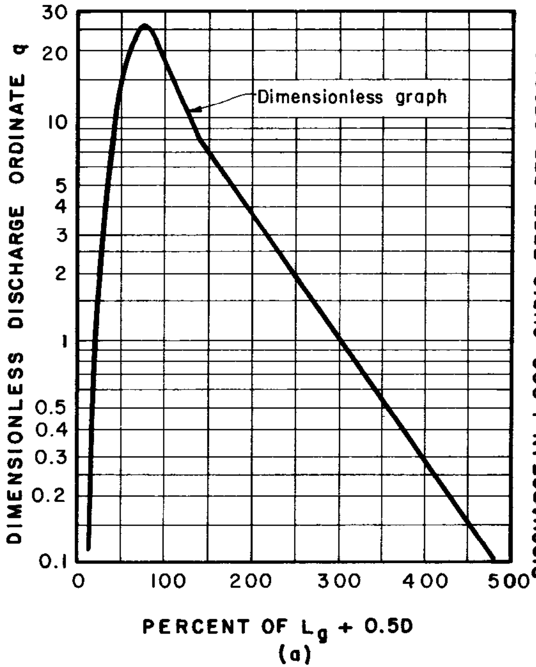
Table 4-18.—Dimensionless S-graph data for urban basins.

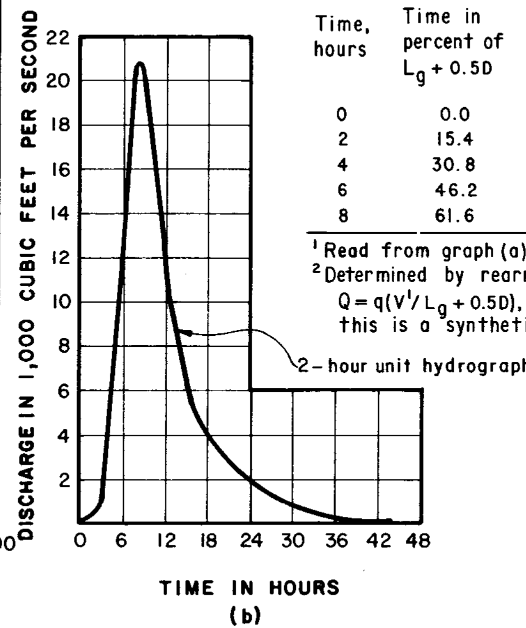
Time *t*, in % of L_g	Discharge, % of ultimate	Time *t*, in % of L_g	Discharge, % of ultimate	Time *t*, in % of L_g	Discharge, % of ultimate	Time *t*, in % of L_g	Discharge, % of ultimate	Time *t*, in % of L_g	Discharge, % of ultimate	Time *t*, in % of L_g	Discharge, % of ultimate
5	0.14	105	52.94	205	82.34	305	92.18	405	96.82	505	99.05
10	.48	110	55.64	210	83.06	310	92.51	410	96.98	510	99.12
15	1.04	115	58.13	215	83.75	315	92.82	415	97.13	515	99.19
20	1.82	120	60.42	220	84.40	320	93.12	420	97.27	520	99.26
25	2.84	125	62.53	225	85.02	325	93.40	425	97.41	525	99.33
30	4.11	130	64.50	230	85.60	330	93.68	430	97.54	530	99.39
35	5.64	135	66.32	235	86.17	335	93.95	435	97.67	535	99.45
40	7.49	140	68.01	240	86.71	340	94.21	440	97.79	540	99.51
45	9.67	145	69.59	245	87.23	345	94.46	445	97.91	545	99.57
50	12.21	150	71.06	250	87.73	350	94.69	450	98.03	550	99.62
55	15.14	155	72.42	255	88.22	355	94.92	455	98.14	555	99.67
60	18.51	160	73.71	260	88.68	360	95.15	460	98.25	560	99.72
65	22.33	165	74.91	265	89.13	365	95.36	465	98.35	565	99.77
70	26.47	170	76.04	270	89.56	370	95.57	470	98.45	570	99.82
75	30.71	175	77.10	275	89.98	375	95.77	475	98.54	575	99.87
80	34.95	180	78.10	280	90.38	380	95.96	480	98.64	580	99.91
85	39.12	185	79.04	285	90.77	385	96.15	485	98.73	585	99.95
90	43.09	190	79.94	290	91.14	390	96.33	490	98.81	590	99.99
95	46.72	195	80.78	295	91.50	395	96.50	495	98.89	595	100.00
100	50.00	200	81.58	300	91.85	400	96.66	500	98.97		

should always be rounded down to the closest of 5, 10, 15, or 30 minutes; or 1, 2, 3, or 6 hours. If the result is greater than 6 hours, the basin should probably be divided into subbasins and an individual unit hydrograph developed for each. This is generally the case for basins exceeding 500 square miles. The runoff hydrographs resulting from the application of rainfall to each subbasin will be routed and combined at the concentration point as discussed in chapter 5, resulting in the flood hydrograph for the total basin.

As previously discussed, the dimensionless unit hydrograph is expressed in terms of time in percent of lag plus the semiduration of unit rainfall on the abscissa, and the ordinate is expressed in terms of discharge multiplied by the value found by adding lag time and semiduration of unit rainfall, and then dividing by the volume of 1 inch of runoff for the basin under study. Since the lag time is known and the volume of 1 inch of runoff can be determined using the drainage basin's area in square miles, the selected dimensionless unit hydrograph can be used to compute the unit hydrograph. This methodology is explained by the following example:

> Consider a 300-square mile drainage basin whose unit hydrograph lag time has been determined to be 12 hours, and that a unit duration of 2 hours (12 hours/5.5 = 2.2 hours, use 2 hours) has been selected for use in developing the unit hydrograph. Also assume that the dimensionless unit hydrograph shown on figure 4-12 has been selected as the basis for developing the unit hydrograph for the basin in question.
>
> The value of lag time plus semiduration of unit rainfall is equal to 12 + 2/2, or 13 hours. The volume of 1 inch of runoff is equal to 300 times the conversion factor 26.89, or 8,067 1-day cubic feet per second. After these values are determined, a table is set up as shown on figure 4-12. The first factor of 26.89 is used to convert 1 inch of rainfall excess over a 1-square mile area in 24 hours to runoff expressed as 1-day cubic feet per second. The first column in the table lists the time in hours, with each increment equal to the unit rainfall duration. Values in the second column, percent of $L_g + 0.5D$ (percent of lag time plus semiduration of unit rainfall), are determined by dividing the corresponding value in the first column by the lag time plus semiduration value, and multiplying by 100 to convert to percent. Values in the third column are obtained by reading the ordinate value from the dimensionless unit hydrograph for the corresponding percent of lag plus semiduration value in the second column. The synthetic unit hydrograph ordinates Q_s resulting from multiplying values in the third column by the quotient of the 1-inch runoff volume divided by the lag plus semiduration value are listed in the fourth column. The ordinates so developed represent the unit discharge at the end of the respective time period.





SYNTHETIC UNIT HYDROGRAPH DERIVATION

Drainage area = 300 square miles
Lag time L_g = 12 hours
Unit duration D = 2 hours
Unit volume of runoff from basin,
$V' = 8{,}067$ 1-day cubic feet per second
$L_g + 0.5D = 13$ hours

Time, hours	Time in percent of $L_g + 0.5D$	Dimensionless unit hydrograph ordinate[1], q	Synthetic unit hydrograph ordinate[2], Q_s
0	0.0	0.0	0
2	15.4	0.4	248
4	30.8	3.1	1,924
6	46.2	9.0	5,584
8	61.6	17.2	10,673

[1] Read from graph (a)
[2] Determined by rearranging equation (3) to yield: $Q = q(V'/L_g + 0.5D)$, and substituting Q_s for Q since this is a synthetic unit hydrograph derivation.

Figure 4-12.—Dimensionless unit hydrograph and sample computations. 103-D-1856.

After the synthetic unit hydrograph ordinates Q_s are determined, they should be plotted on graph paper and a smooth curve drawn through the points. Since the curve will not pass through all the points, the final synthetic unit hydrograph ordinates used in developing a flood hydrograph should reflect values read from the curve rather than the computed values. A graphical representation of the final synthetic unit hydrograph and a tabulation of the final ordinates should be included in every flood study report.

(2) *Dimensionless S-graph technique.*—As was the case with the dimensionless unit hydrograph technique, the unit duration is the first item to be determined. The same constraints apply to this technique in determining the unit duration as applied to the dimensionless unit hydrograph relative to the length of the unit duration and the subdivision of the drainage basin. As previously stated, the dimensionless S-graph is expressed in terms of time in percent of unit hydrograph lag time on the abscissa and discharge as a percentage of ultimate discharge on the ordinate. The ultimate discharge is an equilibrium rate of discharge achieved at the time when the entire drainage basin is contributing runoff at the concentration point from a continuous series of unit rainfall excess increments. The ultimate discharge for a drainage basin is found by multiplying the drainage area, in square miles, by the conversion factor 645.3 and then dividing the result by the unit duration of rainfall. The conversion factor 645.3 represents the volume of 1 inch of runoff from 1 square mile of drainage and is expressed as 1-hour cubic feet per second. With both the lag time and ultimate rate of discharge known, application of these values to the appropriate dimensionless S-graph yields a synthetic unit hydrograph, as described in the following example:

Consider a drainage basin with an area of 250 square miles and a lag time of 12 hours. The theoretical unit duration is then 12/5.5, or 2.18 hours (use 2 hours for computational purposes). The ultimate discharge for this basin from a continuous series of rainfall increments of 1 inch in every 2-hour period would be 250 (646.6)/2, or 80,700 2-hour cubic feet per second. The dimensionless S-graph shown on figure 4-13 is assumed to be appropriate for the hypothetical basin under consideration, and is selected for use in this example. The synthetic unit hydrograph, truncated at the 16th hour for brevity, is then developed as shown in table 4-19.

In table 4-19, time is tabulated in the first column at increments equal to the unit duration. Time is expressed as a percentage of lag time L_g in the second column, and is found by dividing the time value in the first column by the unit hydrograph lag time. Values in the third column represent ordinate values read from the dimensionless S-graph at the corresponding time in percent of lag values shown in second column. Each value in the third column is multiplied by the ultimate discharge

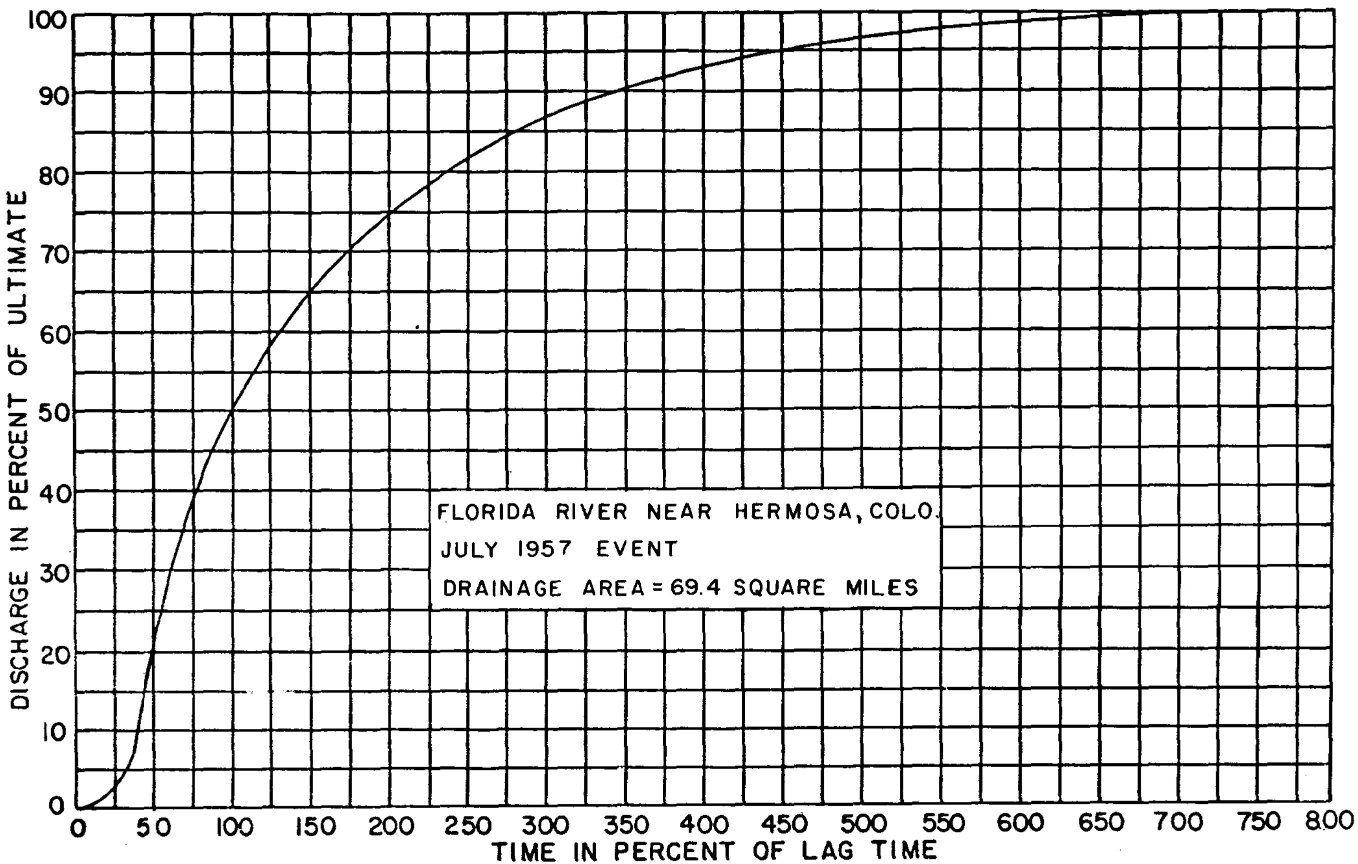


Figure 4-13.—Typical dimensionless S-graph. 103-D-1857.

Table 4-19.—Values for synthetic unit hydrograph.

Time, hours	Time in percent of lag	Discharge in percent of ultimate	Summation hydrograph ordinates, ft³/s	Unit hydrograph ordinates, ft³/s
0	0	0	0	0
2	17	1	807	807
4	33	6	4,840	4,033
6	50	21	16,939	12,099
8	67	35	28,232	11,293
10	83	43	34,685	6,453
12	100	50	40,331	5,646
14	117	55	44,364	4,033
15	125	58	46,784	2,420
16	133	60	48,397	1,613

resulting in the summation hydrograph ordinates shown in fourth column. The resulting synthetic unit hydrograph ordinates in the fifth column are obtained by subtracting successive values from the preceding one. It should be noted that these values represent the ordinates of the synthetic unit hydrograph at the specific time indicated. The Bureau uses the instantaneous end of period ordinates rather than an ordinate representing the average discharge for each unit time period.

The synthetic unit hydrograph ordinates, whether computed manually or by computer, should be plotted on graph paper at the proper time intervals and a smooth curve drawn through the points. The final synthetic unit hydrograph ordinates will reflect the position of the smooth curve rather than the computed values.

(h) Observed Flood Hydrograph Reconstitutions.—From the preceding discussions, it can be seen that the results of analyses of recorded flood events in the form of hydrograph reconstitution [9] provide the primary basis for development of design flood hydrographs used for sizing hydraulic features on Bureau projects. The basic data required for derivation of a unit hydrograph through the reconstitution of an observed hydrograph recorded for a drainage basin tributary to a particular gauging station are as follows:

- Continuous discharge hydrographs for all major flood rises occurring at the gauging station for which adequate corresponding storm precipitation data are available. Except for large drainage areas whose flood hydrographs do not peak rapidly, mean daily discharges as published in the USGS Water Supply Papers are not satisfactory for reconstructing the flood hydrograph. Continuous discharge values should be obtained as outlined in chapter 2.

- Precipitation data for corresponding storms are required for all rainfall stations in and adjacent to the basin, including hourly rainfall data from available recording rain gauges. The stations should be sufficient in number to define the areal distribution of rainfall, and data from sufficient recording gauges should be available to determine the variation of rainfall intensity throughout the storm period.

- A topographic map showing the area of the drainage contributing to streamflow at the stream-gauging station.

(i) Hydrograph Reconstitution Procedure.—The basic "trial and error" process for reconstituting an observed flood hydrograph by the unit hydrograph procedure takes the following steps [9]:

Step 1. On a topographic map (a USGS 7.5-minute quadrangle map for smaller basins and 1:250,000 maps for larger basins), outline the drainage basin boundary, longest watercourse from gauging station site to drainage basin boundary, and location of centroid of the basin. Also, identify each location where precipitation data have been collected for a particular storm with a suitable symbol, usually a cross for stations where a continuous record has been taken and a circle for nonrecording stations or locations where total storm data have been accumulated. Identify each station or location with a number that will be used as a cross reference to a table that is to be prepared listing station name, location of each station (latitude and longitude), and type of station (recording or nonrecording).

Step 2. Determine, by planimeter and opisometer (map measurer) or computerized digitizer: (1) area, in square miles, of drainage basin; (2) length, in miles, of longest watercourse; (3) overall slope of longest watercourse, in feet per mile, from gauging station site to drainage basin boundary; (4) centroid of drainage basin, and (5) distance, in miles, from gauging station along longest watercourse to a point opposite centroid of drainage basin. If a digitizer is not available, the centroid of the basin can be found by suspending a heavy cardboard template of the drainage basin by using a pin near the template edge and drawing a vertical line from the pin across the template. The template is then rotated about 90°, pin is again placed near the template edge, and another vertical line is drawn from the pin across the template. The intersection of these two lines is the centroid of the basin.

Step 3. Determine the basin average precipitation for the total storm. This is accomplished by using either an isohyetal or Thiessen Polygon approach:

a. The isohyetal approach uses contour lines (isohyetal lines) representing equal rainfall amounts. Isohyetal lines representing precipitation values, in whole numbers, are drawn using each station's precipitation data as a base and a straight-line interpolation between station values. When the lines are drawn, the areas between each adjacent line and lying within the drainage basin boundary are determined. Each area has a representative rainfall amount associated with it that is determined by averaging the values of the two isohyetal lines encompassing the area within the drainage basin. The basin average precipitation is then determined by dividing the product of each area and its representative rainfall amount by the total drainage basin area.

b. The Thiessen Polygon approach for determining average basin rainfall for the total storm consists of developing polygons formed by connecting the perpendicular bisectors of lines connecting each of the precipitation stations within and outside of the drainage basin. Sufficient polygons must be developed so that all of the drainage basin is covered. The area within each polygon is determined by planimetering or by using a computerized digitizer. The average basin rainfall is then computed by dividing the product of the total rainfall recorded at the station within each polygon and the area encompassed by that polygon by the total drainage basin area.

Step 4. It has been assumed that at least one of the precipitation stations within or near the drainage basin is of the recording variety. In addition to mechanically or electronically recording the data, manual readings of instruments at recorded time intervals are entirely satisfactory. This station, or stations, will be used to distribute the total storm rainfall by discrete increments of time. The determination of these time increments are dependent on the required unit duration of the unit hydrograph. Since the unit hydrograph has yet to be developed for the case at hand, an estimate of the appropriate unit duration is necessary. As previously discussed, the unit duration of rainfall should approximate 1/5.5 of the unit hydrograph's lag time. The trial unit hydrograph lag time can be estimated for the purpose of a first approximation of observed hydrograph reconstruction by using equation (4) from section 4.1(e).

All of the values in equation (4) are readily obtainable from a USGS 7.5-minute quadrangle map or from a 1:250,000 map, with the exception of the K_n value, which is determined either by intimate knowledge of the drainage basin, by previous studies of nearby basins, or by using an average value from one of the appropriate lag relationships shown on figures 4-6 through 4-11.

Once the unit hydrograph lag time has been approximated, rainfall data that will provide incremental rainfall depths for time increments equal to 15 to 20 percent of the estimated unit hydrograph lag time are used to develop the rainfall hyetograph. A hyetograph is a bar graph indicating the depth of rainfall occurring in a specified time, and is plotted on the graph showing the recorded runoff hydrograph, see figure 4-14. It is imperative that both the hyetograph of basin rainfall and the recorded runoff hydrograph are on the same time base. Many previous flood hydrograph analyses for smaller drainage basins have encountered difficulties simply because the rainfall data were on a daylight savings time base and the runoff data were on a standard time base, or visa versa, and the analyst failed to recognize the fact.

Step 5. Referring to the theory and rationale presented in more detail in the following subsection (j) on "Infiltration and Other Losses," it is now necessary to assign appropriate infiltration losses to the rainfall occurring over the basin. Since the storm and resulting flood being analyzed should be a major event, ideally of 50-year or less frequency, the infiltration rates toward the end of the storm should approach the minimum or ultimate values for the types of soils in the drainage basin after the soils have become saturated. A trial infiltration rate decay curve is superimposed on the hyetograph such that the volume of rainfall excess equals that portion of the recorded hydrograph assumed to represent surface runoff, as discussed in step 6. Incremental values of trial infiltration losses and the resulting rainfall excess are tabulated as shown on figure 4-14.

Step 6. Separate the base flow and interflow components from the recorded runoff hydrograph. Referring to figure 4-14, the dashed line represents the base flow component, which is primarily a function of ground-water supply of perennial streams and may show an essentially constant rate of discharge for some period prior to the flood event. The base flow component may be entirely absent in such cases as the dry arroyos in the desert Southwest or in intermittent streams elsewhere. The interflow component, shown at the bottom of figure 4-14, will always be present to some degree because it represents rapid subsurface flow of infiltrated water to an adjacent watercourse. Because this water travels underground to the watercourse, the elapsed time between rainfall and the resulting appearance as runoff in a watercourse is considerably longer than the surface runoff. The actual position and form of the interflow component for a given flood reconstruction is assumed. The interflow component is one of the three factors that must be adjusted and balanced within reasonable limits to eventually reproduce the recorded flood hydrograph, the other two factors are infiltration loss and unit hydrograph. It should be remembered that the volume of surface runoff, excluding base and interflow, must equal the volume of rainfall excess. Therefore, when adjusting either the infiltration losses or the base and/or interflow components, compensating adjustments must also be made to ensure a reasonable balance between rainfall excess and surface runoff.

Step 7. Construct a trial unit hydrograph. Since the trial unit hydrograph lag time was determined in step 4, the easiest and most reliable

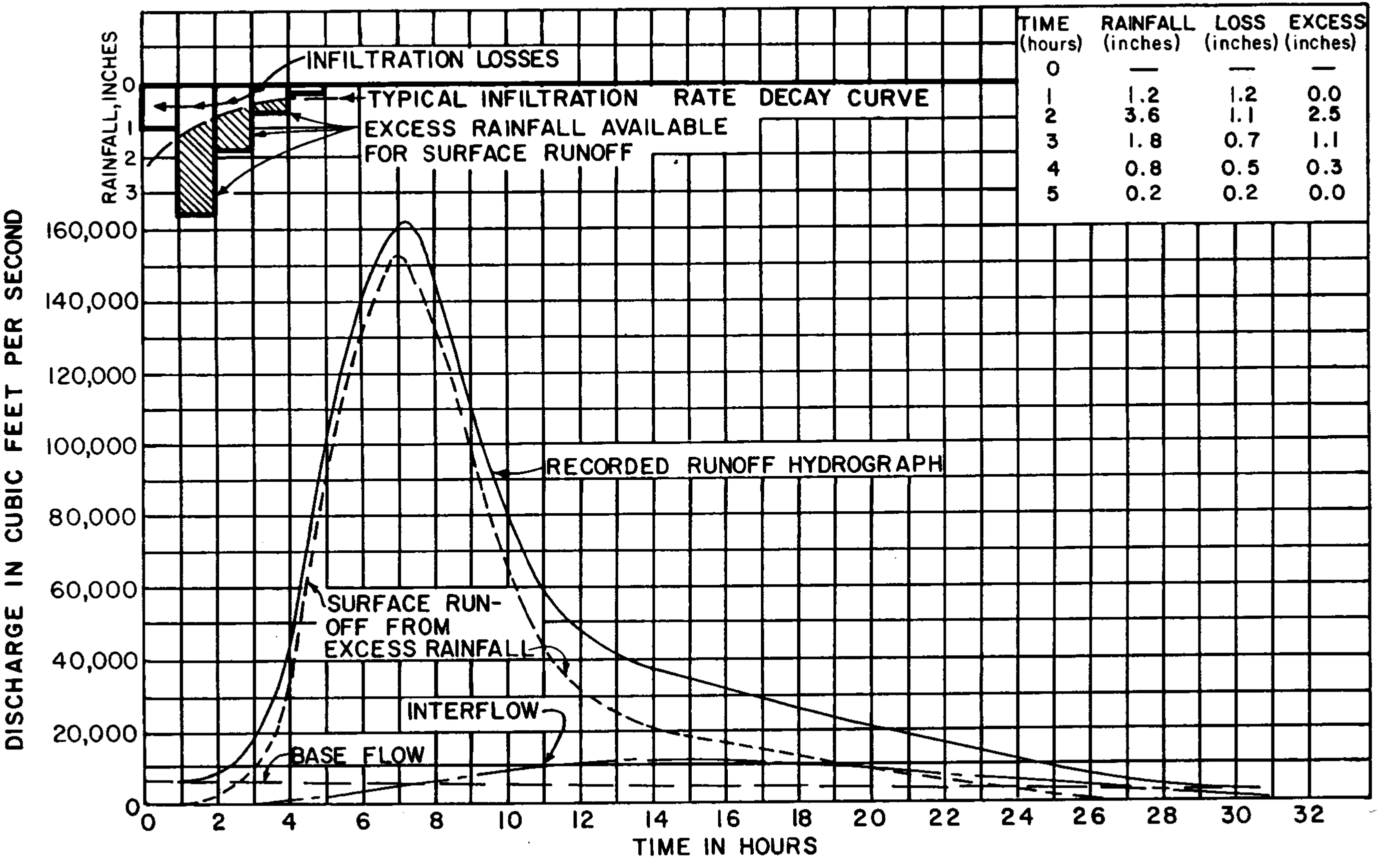


Figure 4-14.—Typical components of total flood runoff hydrograph. 103-D-1858.

method of developing the trial (or "first cut") unit hydrograph is probably by using the S-graph approach previously discussed in section 4.1(g)(2).

Step 8. Apply the trial unit hydrograph to the incremental rainfall excess amounts determined in step 5 by manually arranging the rainfall excess amounts in tabular form. The trial unit hydrograph ordinates are then listed on a strip of paper in the reverse of their actual sequential order. Then, align the strip such that the first ordinate of the unit hydrograph is opposite the first increment of rainfall excess. Multiply theunit hydrograph ordinate by the rainfall excess increment to obtain the resulting surface runoff at that point in time. Move the strip down one time increment and multiply the unit hydrograph ordinate by the adjacent rainfall excess increment, and sum the successive products to obtain the total surface runoff hydrograph ordinate at that time increment. This method is repeated for each successive time increment. Naturally, there are several computer programs available to accomplish this task in a much more rapid and less time consuming manner.

Step 9. It is rare when the analyst successfully reproduces the observed flood hydrograph on the first trial. If it is found that the computed peak is lower than the observed peak, then the trial unit hydrograph should be peaked up by concentrating more volume around the peak. If the computed peak is higher than observed peak, the trial unit hydrograph should be flattened. Occasionally, the peak of the trial hydrograph will occur either earlier or later than the observed hydrograph. If this is the case, the lag time should be adjusted accordingly and a new trial unit hydrograph developed for the next trial at reproducing the observed flood hydrograph. With experience, this apparently tedious process becomes relatively easy to accomplish.

(j) Infiltration and Other Losses. [6].—This subsection deals with the atmospheric processes that result when either rainfall or snowfall become separated into several parts upon reaching the ground surface. The flood hydrologist is primarily concerned with four of these parts:

1. *Interception* by vegetation and subsequent evaporation or retardation when reaching the ground surface.

2. *Evaporation* from ground surface during prolonged rainfall events or when accumulated in frozen form from snowfall, which is generally termed "sublimation."

3. *Retention* in surface depressions that act as miniature reservoirs that do not release their waters until their storage capacity is exceeded, and then only in relation to a stage versus discharge relationship comparable to an uncontrolled spillway on a reservoir.

4. *Infiltration* into the receiving soil, rock, or combination thereof. Any of the constituents of the Earth's mantle have a capability to absorb water, whether it is a concrete airport parking and loading area or the most sandy of soils comprising some areas of the arid West.

The first three of the above loss processes are usually low when compared with infiltration when rainfall intensities are sufficient to produce severe flood events such as the PMF. Under such conditions, the first three are often grouped with part of the infiltration loss and termed "initial losses," and assumed to have been satisfied by antecedent rainfall occurring prior to the onset of the PMS.

To illustrate the phenomena that occurs in the soil when water is applied in the form of rain, consider a condition at the onset of a rainstorm where the soil is comparatively dry as a result of no precipitation having recently occurred. Initially, part of the precipitation is intercepted by vegetation and, after the vegetation has reached its capacity to retain water by surface tension, additional precipitation simply runs off the leaves, stems, etc., and falls to the ground. Also, part of the rainfall falls directly on the ground surface and enters the soil. In nature, some of this precipitation evaporates back into the atmosphere; however, in the hydrologic analysis of floods, interception and evaporation losses are so small compared to the magnitude of the precipitation that they are neglected.

The process of water moving through the ground surface into a soil profile is referred to as "infiltration". The infiltration rate is the "volume per unit area per unit time passing through the ground surface and flowing into the profile." Infiltration rates are sensitive to local conditions at or just below the ground surface and; therefore, the soil wetness, compaction, shrinkage cracks, organic matter, swelling of clay soils, root holes, and animal burrows can all have a marked effect on infiltration rates. However, as the soil profile becomes saturated, conditions deeper within the soil profile become more important and eventually may control the process.

When water is ponded at the surface of a dry soil, gravity and soil-water tension forces act together to move the water downward through the soil profile. Although the hydraulic conductivity is small, the soil-water tension gradients are large, making the infiltration rates high. These rates are controlled by soil profile conditions, and are called the "soil's infiltration capacity." Tension gradients are greater than gravity gradients, and the soil is said to absorb water in the same manner as a blotter absorbs ink.

As the soil profile becomes more saturated, the soil-water tension gradients decrease, hydraulic conductivity increases, and the infiltration capacity decreases in an exponential decay fashion, asymptotically approaching the ultimate infiltration rate. The ultimate or final infiltration rate is theoretically equal to the saturated hydraulic conductivity because tension gradients are no longer present and the hydraulic gradient is due solely to gravity.

When the precipitation rate is less than the soil profile's infiltration capacity, infiltration rates are identical to the precipitation rates and no water is available for surface runoff. Since the infiltrating water alters the water content in the soil profile, the infiltration capacity also changes. At some point during a precipitation event, precipitation rates may exceed the infiltration capacity of the soil, which results in ponding and/or surface runoff. Continued rainfall will then produce the characteristic decay or decrease in infiltration rates. This phenomenon, as it relates to severe flood occurrences, can be represented by a decay curve function where the infiltration capacity rather than rainfall rates control the infiltration rates. In 1940, Horton [58] proposed the following mathematical relationship to represent this function:

$$f = f_c + (f_o - f_c)\, e^{-kt} \tag{5}$$

where:

f = resulting infiltration rate at time t, in hours;
f_c = minimum or ultimate infiltration rate, in inches per hour;
f_o = initial rate of infiltration capacity, in inches per hour;
e = base of natural logarithm;
k = a constant dependent primarily on soil type and vegetation; and
t = time from start of rainfall, in hours.

In the development of the PMF, the hydrologic engineer is primarily concerned with the magnitude of f_c in equation (5).

Many attempts have been made to measure infiltration rates using devices known as infiltrometers. When infiltration rates developed from the infiltrometer tests are compared with those from observed flood hydrograph reconstitutions, the test results from the infiltrometer are almost always higher. Bureau hydrologic engineers consider rates resulting from reconstitution studies to be more valid because they tend to reflect the integrated infiltration rates for the various soil conditions over the entire drainage basin.

The SCS (Soil Conservation Service) has proposed the subdivision of soils into four groups relative to their respective infiltration capacities or ultimate infiltration rates. The ultimate or minimum infiltration rates of these four groups have been found by the Bureau to be in reasonably close agreement with the rates resulting from observed flood hydrograph reconstitution studies. When more than one group of soils is present in a drainage basin, an average value for the basin should be calculated based on weighted areas. The four groups as generally defined by the SCS are:

(1) *Group A soils* (low runoff potential).—Soils that have high infiltration rates even when thoroughly wetted, and consisting mostly of well- to excessively well-drained sands or gravels. These soils have a high rate of water transmission. Ultimate infiltration rates for these soils have been found to range from 0.3 to 0.5 inch per hour.

(2) *Groups B soils.*—Soils having moderate infiltration rates when thoroughly wetted, and consisting mostly of moderately deep to deep, moderately well- to well-drained soils with moderately fine to moderately coarse textures, which would include sandy loams and shallow loess. These soils may also include moderate organic matter. Ultimate infiltration rates for these soils range from 0.15 to 0.30 inch per hour.

(3) *Group C soils.*—Soils having slow infiltration rates when thoroughly wetted, and consisting mostly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission, and include many clay loams, shallow sandy loams, soils low in organic matter, and soils usually high in clay content. The minimum or ultimate infiltration rates for these soils range from 0.05 to 0.15 inch per hour.

(4) *Group D soils* (high runoff potential).—Soils having very slow infiltration rates when thoroughly wetted, and consisting mostly of clay soils with high swelling potential, soils with a permanent high-water table, soils with a claypan (e.g., desert pavement) or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of transmission, and include heavy plastic clays and certain saline soils. Minimum infiltration rates range from 0 to 0.05 inch per hour.

Hydrologic analyses leading to PMF estimates should be based on the assumption that minimum or ultimate infiltration rates prevail throughout the duration of the PMS. This assumption is based on consideration of conditions that have been shown to exist prior to extreme storm events. Examination of historical conditions have shown that it is entirely reasonable to expect one or more storms antecedent (prior) to the extreme event due to meteorologic persistence. Accordingly, it is reasonable to assume that any antecedent storm has satisfied any soil moisture deficiencies and initial losses, and that infiltration rates would be at the minimum or ultimate rate at the onset of the PMS.

(k) Base Flow and Interflow.—The base flow and interflow components to a flood hydrograph are graphically shown on figure 4-14. The base flow component generally consists of the water reaching a basin's watercourses after flowing a considerable distance underground as ground water. The base flow is generally depicted as a recession curve, which indicates a gradually decreasing rate of flow. This flow continues to decrease until the water surface in the stream is in a state of equilibrium with the surface of the adjacent water table and the flow is maintained by inflow from the ground-water reservoir. When the water table is at a level below the channel bed, there will be no surface flow in the stream; however, subsurface flow may be taking place in the river gravels. If this is the case, the recession curve will approach and finally go to zero.

The interflow component, occasionally referred to as the subsurface storm flow, is generated by precipitation that enters the ground by infiltration but emerges as a direct contribution to the surface runoff within a relatively short period. Bureau hydrologic engineers currently believe this phenomenon is present in every severe flood event in varying degrees, depending on the particular drainage basin.

Quantification of the base flow and interflow components in a flood study is usually based on the results of flood hydrograph reconstitutions. The separation of the observed flood hydrograph into the three components shown on figure 4-14 requires a considerable amount of judgment because the interflow and base flow (recession flow) are considerably more indeterminate than the surface flow component.

The magnitude of the base flow is highly dependent on antecedent storm conditions both in terms of that storm's magnitude and the time elapsed between its occurrence and the onset of the storm that produced the flood being analyzed. Providing that sufficient data are available, which is rarely the case, a complete recession curve representing the base flow component for a given drainage basin can be determined. The recession or base flow used in the development of the PMF hydrograph should represent conditions that are consistent with the antecedent storm conditions provided for in the storm study report. For example, a higher recession flow is used when there is a 1-day separation between the antecedent storm and the PMS than would be used if a 5-day separation was assumed. When preparing a flood study for an ungauged watershed, results of observed flood reconstitutions on hydrologically similar drainage basins relative to the base flow component are used to estimate this component for the ungauged basin. This may be accomplished by converting the observed component to cubic feet per second per square mile of drainage basin area. The result is then applied to the area of ungauged drainage basin under study to determine an appropriate rate of base flow for that basin. It is entirely proper to assume that the base or recession discharge rate is uniform for the duration of the PMF hydrograph.

The interflow component is determined essentially by a trial and error approach in the course of observed flood hydrograph reconstitutions previously discussed. After subtracting the base or recession flow component, the remaining observed flood hydrograph is composed of the surface flow and interflow components. When separating these latter two components, care must be taken to ensure that neither too much nor too little flow is assigned to the interflow component. A reasonable balance can be achieved by the adequate selection of infiltration loss rates previously discussed.

When an ungauged watershed is being studied, interflow information from observed flood hydrograph reconstructions for hydrologically similar nearby watersheds are used to estimate the magnitude and rate of change of discharge over time. As was the case for the base flow component, the conversion from the observed hydrograph to that for the ungauged basin is based on a direct ratio of the respective drainage basin areas. The resulting interflow hydrograph should incorporate a rising limb, a rather broad peak, and a long recession limb.

4.2 Probable Maximum Flood Hydrographs

The Bureau's definition of the PMF (probable maximum flood) hydrographic is "the maximum runoff condition resulting from the most severe combination of hydrologic and meteorologic conditions that are considered reasonably possible for the drainage basin under study." Accordingly, since the unit hydrograph approach is used to develop the design flood hydrograph, the following considerations should be followed when computing the hydrograph:

1. The PMF will be based on the PMP or storm values developed using the criteria furnished in the appropriate HMR discussed in chapter 3. The temporal distribution of the storm rainfall will be consistent with the hydrometeorological report criteria. In the absence of such criteria, the storm rainfall values will be arranged in such a manner that the maximum peak discharge and the maximum concentration of discharge around the peak is achieved, as discussed in chapter 3.

2. Infiltration rates that are subtracted from the storm rainfall to obtain the excess amount available for surface runoff should be at minimum or ultimate rates consistent with the soil types and underlying geologic conditions of the drainage basin under study. These infiltration rates should also be consistent with the PMF concept, and should be more conservative than values reflected by averages. These minimum or ultimate rates will be assumed to prevail throughout the duration of the PMS event.

3. The unit hydrograph used to compute the PMF should be representative of extreme discharge conditions. In situations where studies are being prepared for gauged basins for which results of observed flood hydrograph analyses are available, care should be taken to ensure the unit hydrograph parameters adequately reflect conditions likely to occur in a probable maximum event. It is entirely appropriate to modify the K_n value in the general unit hydrograph lag equation in either an upward or downward direction to reflect the decreased or increased hydraulic efficiency of the drainage basin's network that would be expected in association with the high discharges that occur during such an extreme event. When a flood study involves an ungauged basin, considerable judgment must be used to ensure that the

K_n values approximate the basin's expected hydraulic efficiency as closely as possible.

4. The base or recession flow hydrograph component should reflect the maximum rates of discharge that are consistent with the magnitude and timing of any antecedent flood event.

5. The interflow component should reflect the optimum condition that could be expected to result from a PMS event. However, this component will probably not be significantly different than would be experienced in a relatively minor event because the hydraulic efficiency of the subsurface media through which this component passes is essentially fixed.

Frequently, a rain-on-snow PMF hydrograph is desired. The basis and rationale for adding a snowmelt runoff component to the rainflood hydrograph are discussed in section 4.3.

4.3 Western Mountain Snowmelt Equation

A method that has been used by the Bureau has become known as the "Snow Compaction Method." This method is described in detail in the Bureau's *Engineering Monograph No. 35* [59]. Proper application of this method requires either data or assumed values for air temperatures, wind speeds, percent of forest cover, depth of snow, and density of snow at various elevation bands. The wind speeds and air temperatures are usually furnished by Bureau meteorologists as part of the PMS study when it is anticipated that the snowmelt runoff will contribute to the PMP to generate the PMF. From a record search, the hydrologist determines snow depths and densities that are considered reasonable for initial watershed conditions at the onset of the PMS. In most cases, the drainage basin is divided into elevation bands for analysis. Depending on the size of the basin and the elevation variation within the basin, the elevation bands are usually selected at 500- or 1,000-foot intervals. For those basins that are relatively flat, the entire basin may be used as one elevation band. The rain-on-snow contribution from each elevation band is computed and averaged over the total basin, and then added together to yield the total area runoff.

This method requires several assumptions by the hydrologic engineer because there are usually several trial arrangements of rainfall, wind speeds, and air temperatures required to assure the largest flood has been computed. The initial snow depth and density may also have to be adjusted to assure that a reasonable amount of snow has melted and only a moderate amount of rain has been trapped in the remaining snow of the upper elevation bands. Without experience and good judgment, this method can become quite capricious. Currently, there are no consistent, acceptable criteria for determining reasonable values for the several parameters in the snow compaction method. This problem has been a topic of discussion among representatives of the Bureau NWS, COE, and SCS, and future action of this group will be the development of suitable criteria.

In view of this lack of criteria, the standard practice of the Bureau is to combine the rain generated part of the PMF hydrograph with a snowmelt that could reasonably be expected to occur at the time of year that the probable maximum rainfall occurs. Naturally, this practice is only used in those areas where snowpacks of sufficient magnitude to be considered do occur. It should be noted that since the melting snowpack tends to satisfy the infiltration losses, the losses to the rainfall increment are minimal. Current practice is to apply a loss of 0.05 inch per hour to the PMP when generating the PMF rainflood hydrograph. Such losses apply only to the area assumed to be covered by the snowpack.

The Bureau currently uses a 100-year snowmelt flood to account for snowpack. A frequency analysis of the maximum annual snowmelt volume is performed using the procedures discussed in chapter 7. The normal period of runoff selected is 15 days; however, in large drainage basins with significant areas where snowpack accumulates, this period may extend to 30 or 60 days or, in the case of the Colorado River above Hoover Dam, the period may extend through two yearly runoff seasons. The 100-year snowmelt flood is then distributed over time using either the largest snowmelt flood of record as the basis for distribution or by using the balanced flood hydrograph approach as discussed in chapter 7. The resulting snowmelt hydrograph is generally expressed in terms of mean daily flows for the 15-day period with diurnal fluctuations being neglected.

For drainage basins less than about 3,000 square miles, the rainflood hydrograph is superimposed on the snowmelt hydrograph with the rain assumed to occur during the day or days of the greatest snowmelt. This assumption is made so that the maximum rain occurs during the warmest period. The resulting combined rain-on-snow flood represents the probable maximum event. For larger basins, the melting snowpack may form the antecedent flood discussed in section 4.4.

4.4 Antecedent Floods

When developing a PMF hydrograph, it is normal practice to consider antecedent conditions, both meteorologic and hydrologic. The resulting flood hydrograph, in combination with the PMF hydrograph, is generally referred to as the PMF series. The concept of an antecedent event is based primarily on the meteorological factors discussed in chapter 3. The occurrence of antecedent precipitation, either in the form of rain or snow, is the basis for assuming wet or saturated ground conditions and adopting minimum or ultimate infiltration losses in developing PMF hydrographs.

Currently and with two exceptions, none of the Federal water resource development agencies have criteria that reflect definitive hydrologic and hydrometeorologic antecedent storm and flood studies. The two exceptions are antecedent precipitation criteria adopted by the Bureau and COE for the State of Texas, and by the Tennessee Valley Authority for the Tennessee River Basin. Meteorological studies leading to the development of these criteria were performed by the Hydrometeorological and Special Studies Branch, Office of Hydrology, National Weather Service. Documentation of the studies and the resulting criteria are contained in HMR 56, "Probable Maximum and TVA Precipitation Estimates With Areal Distribution for Tennessee River Drainages Less than 3,000 Square Miles in Area" [60], and NWS Technical Memorandum "Precipitation Antecedent to the 24-Hour Probable Maximum Precipitation for Small Basins in Texas"[61].

Since no similar criteria other than the Texas criteria are currently available for the 17 Western States where the Bureau has primary interest, certain provisional criteria have been adopted as established by hydrometeorlogists and hydrologic engineers who have given specific preliminary consideration to the governing meteorlogical factors. Therefore, until such time as definitive studies are conducted, most probably in an interagency effort, the following criteria are used by the Bureau:

(a) For PMF's generated by general PMP events in areas east of the Sierra Nevada and Cascade Ranges, excluding areas covered by the previously mentioned NWS reports, the antecedent flood is estimated by either converting 100-year precipitation to a flood hydrograph or developing a balanced 100-year hydrograph using statistical analyses of runoff data. If 100-year precipitation method is used, the time between the end of antecedent rainfall and the beginning of the PMP event is assumed to be 3 days. If the balanced flood hydrograph approach is used, a time interval of 3 days between the peak of the antecedent flood hydrograph and the beginning of the PMP event is used.

(b) For PMF's generated by general PMP events west of the Sierra Nevada and Cascade Ranges, the same criteria given in (a) apply except that the time intervals are 2 rather than 3 days.

(c) For PMF's generated by local PMP events in the entire region west of the 103rd meridan, no antecedent event is used. Meteorological conditions are such in this region that hydrometeorologists do not consider it reasonable to assume an event of 100-year magnitude to precede the probable maximum event. However, it is reasonable to assume that a storm of sufficient magnitude has occurred to satisfy initial infiltration losses and provide for minimum or ultimate infiltration loss conditions at the onset of the probable maximum local storm event.

(d) For PMF's generated by PMP falling on a snowpack, the antecedent meteorologic condition is assumed to be the snowfall that accumulates and forms the snowpack. It is further assumed that runoff resulting from the melting of the snowpack will occur during and somewhat prior to the probable maximum rainfall. For the majority of drainage basins, those with areas smaller than about 3,000 square miles, current practice is to develop a 100-year balanced snowmelt hydrograph and then superimpose the probable maximum rainflood onto it in such a manner that the peak flow of each hydrograph occurs at the same point in time. For basins larger than 3,000 square miles, it has been frequently found that the snowmelt will peak earlier than it is meteorologically reasonable for the probable maximum rainstorm to occur. In such situations, an evaluation of the timing of the two events should be requested of and performed by experienced hydrometeorologists. Application of the western snowmelt equation discussed in section 4.3(a) is not currently used. However, when definitive criteria for application of this equation have been developed and generally accepted among the Federal water resource development agencies, it is expected that the equation will replace the 100-year balanced hydrograph approach.

4.5 Foss Dam Example

As an example of probable maximum flood hydrograph determinations, consider the Washita River Basin above Foss Dam in Oklahoma. As shown on figure 4-15, this dam and its 1,490-square mile tributary drainage basin are located in west-central Oklahoma, with the basin centroid located at latitude 35°40′N. and longitude 99°45′ W. The upper reaches of this drainage basin extend about 30 miles into Texas. For hydrologic reasons, the basin was divided into four subbasins as shown on figure 4-15. This subdivision will result in the generation of four component flood hydrographs that will require routing and combining to obtain the total flood hydrograph at the dam. This routing and combing will continue this example in chapter 5.

Probable maximum storm precipitation values were obtained for this basin using procedures described comprehensively in HMR 51 and 52 [35,18]. The process for obtaining these values is identical with Example No. 1a shown on pages 108 through 126 of HMR 52. For brevity, the actual PMP computations for the drainage basin above Foss Dam are not included in this manual; however, the results of the computations for the four subbasins are shown in table 4-20.

The first step in the rainfall-to-runoff conversion process is to derive unit hydrographs for each of the four subbasins shown on figure 4-15. The salient physical properties shown in table 4-20 for each subbasin were determined using a USGS quadrangle map.

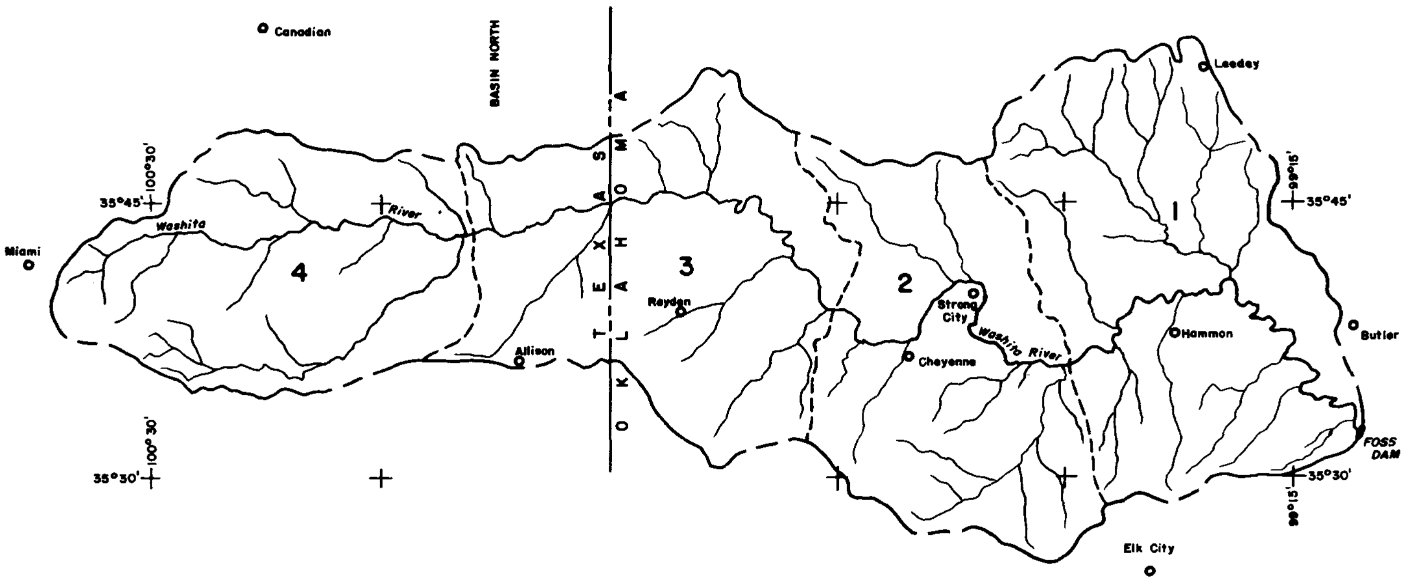


Figure 4-15.—Basin outline for Foss Dam. 103-D-1918.

Table 4-20.—Properties of subbasins in Washita River Basin.

Subbasin No.	Drainage Area, mi^2	L, miles	L_{ca}, miles	S, ft/mi	$\frac{LL_{ca}}{S^{0.5}}$
1	429	26.0	8.2	26.5	41.4
2	333	26.9	11.2	23.1	62.7
3	389	41.3	19.2	17.6	189.0
4	315	32.1	14.0	20.6	99.0

As a result of a field reconnaissance of the Washita River drainage basin tributary to Foss Dam, a factor was assigned for use in the general unit hydrograph lag equation, a suitable dimensionless unit hydrograph selected, and an appropriate infiltration loss rate was determined. Because of the uniformity of the basin, a single factor of 1.8 was assigned to each of the four subbasins. This corresponds to the upper curve shown on figure 4-6. The value of 1.8 reflects a K_n value of 0.069, which is considered characteristic of the drainage network's hydraulic characteristics at extreme flood levels. The unit hydrograph lag time for each subbasin is shown in table 4-21.

The new Arbuckle dimensionless unit hydrograph was selected for distributing the unit runoff over time because this hydrograph was derived from a major recorded flood on a nearby similar drainage basin, and was considered to be more suitable than the Great Plains dimensionless graph for this case. It should be noted that it is entirely proper to substitute a dimensionless graph based on a flood hydrograph reconstitution for the regionalized graphs previously discussed.

Using the shortest unit hydrograph lag time of 6.15 hours for subbasin 1, the unit duration is determined by dividing the lag time by 5.5, or $6.15/5.5 = 1.12$ hours. Thus, a unit duration of 1 hour is adopted. Even though a longer unit duration might be used for subbasin 3, it is always best to retain a single unit duration for all subbasin computations to avoid confusion during the routing and combining process discussed in chapter 5.

All the required information is now known to derive a unit hydrograph for each subbasin. For brevity, only the unit hydrograph for subbasin 1 will be derived in detail. Table 4-22 shows the details of the unit hydrograph development for subbasin 1.

Table 4-22 shows the time, in hours, in the first column, which is the interval corresponding to the unit duration, in this case, 1 hour. The second column records what percent the value in the first column is of the lag plus semiduration of unit rainfall, which is $6.15 + 0.5 = 6.65$ hours for this example. The third column records the value read or

Table 4-21.—Unit hydrograph lag time for each subbasin in Washita River Basin.

Subbasin No.	C_t Coefficient	K_n	$\left(\frac{LL_{ca}}{S^{0.5}}\right)^{0.33}$	L_g, hours
1	1.8	0.069	3.42	6.15
2	1.8	.069	3.92	7.05
3	1.8	.069	5.64	10.15
4	1.8	.069	4.56	8.20

interpolated from the Arbuckle dimensionless graph for values corresponding to lag plus semiduration values for each time increment. The fourth column records the unit hydrograph ordinate for each time increment, and is found by multiplying the total volume of 1 inch of runoff in 1-day cubic feet per second by the ordinate shown in third column divided by the value of the lag plus semiduration. The volume of 1 inch of runoff for the 429-square mile drainage basin is found by multiplying the drainage area by 26.89, which results in 11,536 1-day cubic feet per second.

If the unit hydrograph is computed manually, a check computation should be made. This can be done by totaling the unit hydrograph ordinates in table 4-22 and comparing this total with the volume of 1 inch of runoff from the basin. For this example, the total of the ordinates is 276,846 1-hour cubic feet per second. The volume of runoff from 1 inch of rainfall excess falling in 1 hour is found by multiplying the drainage area, 429 square miles, by a factor of 645.33 which yields 276,846; therefore, the computations check.

After a similar procedure is followed to develop the unit hydrographs for each of the other three subbasins, the next step is to determine the effective rainfall, or the rainfall that is available for surface runoff. This is accomplished by subtracting the infiltration losses from the probable maximum rainfall. As a result of the field reconnaissance of the Washita River Basin above Foss Dam, it was determined that an ultimate infiltration rate of 0.30 inches per hour would be appropriate.

At this point, tables are set up to facilitate both the computation and documentation of the flood hydrograph development, as shown by tables 4-23 through 4-26. In these tables, the first column lists the time increment, and the incremental probable maximum precipitation values in the second column. The third column lists the infiltration losses; e.g., 0.30 inch per hour. Note that the loss cannot exceed the input rainfall; i.e., when rainfall is less than 0.30 inch per hour, the loss equals the rainfall. The fourth column provides the rainfall excess amounts that are available for surface runoff; these are the values that are applied to the unit hydrograph in the manner previously described to obtain the surface runoff hydrograph. The unit hydrograph and resulting surface runoff

Table 4-22.—Unit hydrograph development for subbasin 1.

Time, hours	Percent of $L_g + 0.5D$	Ordinate q from Arbuckle dimensionless graph	Unit hydrograph ordinate Q_s, 1-hour ft^3/s
1	15	0.93	1,615
2	30	3.98	6,896
3	45	10.52	18,426
4	60	20.35	35,298
5	75	25.59	44,387
6	90	16.60	28,786
7	105	10.87	18,860
8	120	9.17	15,900
9	135	7.99	13,859
10	150	6.96	12,078
11	165	6.07	10,525
12	180	5.29	9,182
13	195	4.61	7,998
14	210	4.03	6,988
15	225	3.51	6,081
16	230	3.05	5,298
17	255	2.66	4,618
18	270	2.32	4,027
19	285	2.03	3,521
20	300	1.77	3,070
21	315	1.54	2,670
22	330	1.34	2,321
23	345	1.17	2,027
24	360	1.02	1,768
25	375	0.89	1,542
26	390	.78	1,349
27	405	.68	1,177
28	420	.59	1,019
29	435	.52	897
30	450	.45	778
31	465	.39	672
32	480	.34	589
33	495	.30	520
34	510	.27	467
35	525	.24	415
36	540	.21	362
37	555	.18	309
38	570	.16	275
39	585	.14	242
40	600	.12	214
			Total 276,846

Table 4-23.—Flood hydrograph development for subbasin 1.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft³/s	Hydrograph, ft³/s
1	0.058	0.058	0	1,615	0
2	.060	.060	0	6,896	0
3	.061	.061	0	18,246	0
4	.064	.064	0	35,298	0
5	.065	.065	0	44,387	0
6	.067	.067	0	28,786	0
7	.068	.068	0	18,860	0
8	.069	.069	0	15,900	0
9	.070	.070	0	13,859	0
10	.072	.072	0	12,078	0
11	.073	.073	0	10,525	0
12	.077	.077	0	9,182	0
13	.079	.079	0	7,998	0
14	.083	.083	0	6,988	0
15	.084	.084	0	6,081	0
16	.088	.088	0	5,298	0
17	.090	.090	0	4,618	0
18	.094	.094	0	4,027	0
19	.096	.096	0	3,521	0
20	.099	.099	0	3,070	0
21	.101	.101	0	2,670	0
22	.106	.106	0	2,321	0
23	.109	.109	0	2,027	0
24	.117	.117	0	1,768	0
25	.121	.121	0	1,542	0
26	.129	.129	0	1,349	0
27	.133	.133	0	1,177	0
28	.141	.141	0	1,019	0
29	.145	.145	0	897	0
30	.155	.155	0	778	0
31	.161	.161	0	672	0
32	.175	.175	0	589	0
33	.182	.182	0	520	0
34	.200	.200	0	467	0
35	.210	.210	0	415	0
36	.234	.234	0	362	0
37	.245	.245	0	309	0
38	.278	.278	0	275	0
39	.299	.299	0	242	0
40	.339	.300	0.039	214	63
41	.355	.300	.055	—	358
42	.439	.300	.139	—	1,315
43	.507	.300	.207	—	3,673
44	.901	.300	.601	—	8,607
45	1.274	.300	.974	—	17,965
46	1.760	.300	1.460	—	35,840
47	1.960	.300	1.660	—	66,580

Table 4-23.—Flood hydrograph development for subbasin 1 - Continued.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft³/s	Hydrograph, ft³/s
48	3.100	0.300	2.800	—	113,660
49	1.425	.300	1.125	—	170,830
50	0.592	.300	0.292	—	228,400
51	.388	.300	.088	—	270,600
52	.323	.300	.023	—	273,900
53	.260	.260	0	—	222,100
54	.222	.222	0	—	170,030
55	.190	.190	0	—	138,440
56	.169	.169	0	—	117,690
57	.150	.150	0	—	101,650
58	.137	.137	0	—	88,420
59	.125	.125	0	—	77,080
60	.113	.113	0	—	67,190
61	.103	.103	0	—	58,590
62	.098	.098	0	—	51,060
63	.093	.093	0	—	44,510
64	.086	.086	0	—	38,800
65	.081	.081	0	—	33,840
66	.075	.075	0	—	29,520
67	.071	.071	0	—	25,730
68	.069	.069	0	—	22,410
69	.066	.066	0	—	19,525
70	.063	.063	0	—	17,030
71	.059	.059	0	—	14,857
72	.058	.058	0	—	12,960
73	—	—	—	—	11,310
74	—	—	—	—	9,864
75	—	—	—	—	8,589
76	—	—	—	—	7,501
77	—	—	—	—	6,546
78	—	—	—	—	5,719
79	—	—	—	—	5,025
80	—	—	—	—	4,425
81	—	—	—	—	3,904
82	—	—	—	—	3,415
83	—	—	—	—	2,954
84	—	—	—	—	2,464
85	—	—	—	—	1,983
86	—	—	—	—	1,473
87	—	—	—	—	987
88	—	—	—	—	343
89	—	—	—	—	90
90	—	—	—	—	24
91	—	—	—	—	5
92	—	—	—	—	0

Table 4-24.—Flood hydrograph development for subbasin 2.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft³/s	Hydrograph, ft³/s
1	0.064	0.064	0	897	0
2	.065	.065	0	3,474	0
3	.066	.066	0	9,236	0
4	.068	.068	0	18,123	0
5	.068	.068	0	30,407	0
6	.070	.070	0	26,784	0
7	.072	.072	0	17,969	0
8	.075	.075	0	12,717	0
9	.076	.076	0	10,966	0
10	.079	.079	0	9,712	0
11	.080	.080	0	8,612	0
12	.083	.083	0	7,632	0
13	.084	.084	0	6,760	0
14	.087	.087	0	5,986	0
15	.089	.089	0	5,308	0
16	.093	.093	0	4,710	0
17	.095	.095	0	4,166	0
18	.099	.099	0	3,691	0
19	.102	.102	0	3,274	0
20	.106	.106	0	2,900	0
21	.109	.109	0	2,571	0
22	.115	.115	0	2,272	0
23	.118	.118	0	2,022	0
24	.124	.124	0	1,787	0
25	.127	.127	0	1,580	0
26	.135	.135	0	1,404	0
27	.139	.139	0	1,243	0
28	.149	.149	0	1,100	0
29	.154	.154	0	973	0
30	.166	.166	0	864	0
31	.172	.172	0	771	0
32	.186	.186	0	675	0
33	.193	.193	0	605	0
34	.211	.211	0	535	0
35	.223	.223	0	472	0
36	.249	.249	0	421	0
37	.261	.261	0	369	0
38	.298	.298	0	338	0
39	.323	.300	0.023	306	21
40	.370	.300	.070	275	143
41	.391	.300	.091	243	537
42	.492	.300	.192	211	1,552
43	.573	.300	.273	192	3,720
44	1.061	.300	.761	170	7,798
45	1.537	.300	1.237	152	14,809
46	2.180	.300	1.880	24	27,780
47	2.450	.300	2.150	—	49,900
48	4.420	.300	4.120	—	86,990
49	1.732	.300	1.432	—	136,740

Table 4-24.—Flood hydrograph development for subbasin 2 - Continued.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft³/s	Hydrograph, ft³/s
50	0.673	0.300	0.373	—	193,540
51	.431	.300	.131	—	243,300
52	.351	.300	.051	—	276,200
53	.278	.278	0	—	250,500
54	.236	.236	0	—	197,500
55	.201	.201	0	—	155,220
56	.179	.179	0	—	130,260
57	.160	.160	0	—	113,020
58	.144	.144	0	—	99,340
59	.131	.131	0	—	87,820
60	.121	.121	0	—	77,800
61	.112	.112	0	—	68,930
62	.104	.104	0	—	61,090
63	.097	.097	0	—	54,150
64	.091	.091	0	—	47,970
65	.086	.086	0	—	42,500
66	.082	.082	0	—	37,660
67	.078	.078	0	—	33,370
68	.073	.073	0	—	29,570
69	.069	.069	0	—	26,190
70	.067	.067	0	—	23,220
71	.065	.065	0	—	20,560
72	.063	.063	0	—	18,206
73	—	—	—	—	16,137
74	—	—	—	—	14,299
75	—	—	—	—	12,665
76	—	—	—	—	11,224
77	—	—	—	—	9,954
78	—	—	—	—	8,837
79	—	—	—	—	7,819
80	—	—	—	—	6,946
81	—	—	—	—	6,164
82	—	—	—	—	5,472
83	—	—	—	—	4,879
84	—	—	—	—	4,349
85	—	—	—	—	3,903
86	—	—	—	—	3,495
87	—	—	—	—	3,098
88	—	—	—	—	2,719
89	—	—	—	—	2,315
90	—	—	—	—	1,914
91	—	—	—	—	1,469
92	—	—	—	—	1,031
93	—	—	—	—	415
94	—	—	—	—	123
95	—	—	—	—	37
96	—	—	—	—	11
97	—	—	—	—	1
98	—	—	—	—	0

Table 4-25.—Flood hydrograph development for subbasin 3—Continued.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft³/s	Hydrograph, ft³/s
50	0.719	0.300	0.419	367	118,900
51	.457	.300	.157	340	169,830
52	.368	.300	.068	313	222,100
53	.290	.290	0	291	265,200
54	.248	.248	0	273	291,500
55	.211	.211	0	254	278,300
56	.187	.187	0	236	232,500
57	.166	.166	0	217	187,560
58	.151	.151	0	198	154,970
59	.138	.138	0	180	134,630
60	.125	.125	0	164	121,210
61	.114	.114	0	153	110,530
62	.109	.109	0	141	101,260
63	.102	.102	0	132	92,900
64	.095	.095	0	120	85,280
65	.089	.089	0	—	78,250
66	.085	.085	0	—	71,840
67	.081	.081	0	—	65,940
68	.075	.075	0	—	60,530
69	.070	.070	0	—	55,560
70	.070	.070	0	—	51,020
71	.068	.068	0	—	46,820
72	.067	.067	0	—	42,980
73	—	—	—	—	39,440
74	—	—	—	—	36,190
75	—	—	—	—	33,210
76	—	—	—	—	30,490
77	—	—	—	—	27,990
78	—	—	—	—	25,680
79	—	—	—	—	23,590
80	—	—	—	—	21,640
81	—	—	—	—	19,860
82	—	—	—	—	18,234
83	—	—	—	—	16,748
84	—	—	—	—	15,366
85	—	—	—	—	14,102
86	—	—	—	—	12,952
87	—	—	—	—	11,878
88	—	—	—	—	10,894
89	—	—	—	—	10,006
90	—	—	—	—	9,198
91	—	—	—	—	8,443
92	—	—	—	—	7,733
93	—	—	—	—	7,112
94	—	—	—	—	6,513
95	—	—	—	—	5,982
96	—	—	—	—	5,494
97	—	—	—	—	5,042
98	—	—	—	—	4,657

Table 4-25.—Flood hydrograph development for subbasin 3—Continued.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft³/s	Hydrograph, ft³/s
99	—	—	—	—	4,311
100	—	—	—	—	4,003
101	—	—	—	—	3,720
102	—	—	—	—	3,447
103	—	—	—	—	3,178
104	—	—	—	—	2,912
105	—	—	—	—	2,657
106	—	—	—	—	2,408
107	—	—	—	—	2,178
108	—	—	—	—	1,918
109	—	—	—	—	1,620
110	—	—	—	—	1,265
111	—	—	—	—	892
112	—	—	—	—	281
113	—	—	—	—	81
114	—	—	—	—	28
115	—	—	—	—	8
116	—	—	—	—	0

hydrograph are listed in the fifth and sixth columns, respectively. In this example, both the base flow and interflow were neglected because they would be insignificant as compared to the surface runoff. If these two components had been significant and taken into account, two more columns would be added to the tables. A seventh column would list the combined base flow and interflow, and an eighth column would list the sum of surface runoff, base flow, and interflow.

This example has reached the point where the concurrent probable maximum flood hydrographs have been developed for each of the four subbasins above Foss Dam. These flood hydrographs will now require routing and combining to obtain the inflow flood hydrograph. This subject will be addressed in chapter 5, and this example will then be continued.

Table 4-26.—Flood hydrograph development for subbasin 4.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft³/s	Hydrograph, ft³/s
1	0.053	0.053	0	593	0
2	.055	.055	0	2,053	0
3	.056	.056	0	5,332	0
4	.059	.059	0	10,642	0
5	.060	.060	0	17,783	0
6	.062	.062	0	27,563	0
7	.063	.063	0	21,293	0
8	.063	.063	0	15,096	0
9	.067	.067	0	10,853	0
10	.069	.069	0	9,302	0
11	.070	.070	0	8,379	0
12	.072	.072	0	7,548	0
13	.073	.073	0	6,793	0
14	.075	.075	0	6,120	0
15	.077	.077	0	5,509	0
16	.080	.080	0	4,956	0
17	.082	.082	0	4,462	0
18	.086	.086	0	4,021	0
19	.088	.088	0	3,626	0
20	.092	.092	0	3,265	0
21	.094	.094	0	2,933	0
22	.099	.099	0	2,643	0
23	.102	.102	0	2,379	0
24	.108	.108	0	2,144	0
25	.111	.111	0	1,930	0
26	.118	.118	0	1,741	0
27	.122	.122	0	1,563	0
28	.131	.131	0	1,410	0
29	.135	.135	0	1,264	0
30	.145	.145	0	1,141	0
31	.151	.151	0	1,028	0
32	.163	.163	0	927	0
33	.168	.168	0	833	0
34	.184	.184	0	754	0
35	.194	.194	0	678	0
36	.217	.217	0	611	0
37	.229	.229	0	544	0
38	.260	.260	0	496	0
39	.279	.279	0	445	0
40	.314	.300	0.014	400	8
41	.328	.300	.028	358	45
42	.403	.300	.103	326	193
43	.465	.300	.165	293	608
44	.821	.300	.521	271	1,744
45	1.157	.300	.857	249	4,438
46	1.590	.300	1.290	226	9,960
47	1.740	.300	1.440	204	20,200
48	2.490	.300	2.190	182	36,830
49	1.292	.300	0.992	162	61,590

Table 4-26.—Flood hydrograph development for subbasin 4—Continued.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft^3/s	Hydrograph, ft^3/s
50	0.543	0.300	0.243	147	90,820
51	.357	.300	.057	135	119,490
52	.300	.300	0	120	139,370
53	.244	.244	0	21	147,610
54	.206	.206	0	—	129,170
55	.175	.175	0	—	102,610
56	.157	.157	0	—	81,480
57	.140	.140	0	—	68,430
58	.126	.126	0	—	60,260
59	.114	.114	0	—	54,000
60	.105	.105	0	—	48,590
61	.097	.097	0	—	43,750
62	.090	.090	0	—	39,390
63	.084	.084	0	—	35,470
64	.079	.079	0	—	31,940
65	.074	.074	0	—	28,770
66	.071	.071	0	—	25,910
67	.068	.068	0	—	23,330
68	.064	.064	0	—	21,000
69	.061	.061	0	—	18,906
70	.058	.058	0	—	17,024
71	.054	.054	0	—	15,333
72	.053	.053	0	—	13,804
73	—	—	—	—	12,430
74	—	—	—	—	11,184
75	—	—	—	—	10,069
76	—	—	—	—	9,062
77	—	—	—	—	8,164
78	—	—	—	—	7,356
79	—	—	—	—	6,629
80	—	—	—	—	5,969
81	—	—	—	—	5,378
82	—	—	—	—	4,842
83	—	—	—	—	4,358
84	—	—	—	—	3,917
85	—	—	—	—	3,532
86	—	—	—	—	3,183
87	—	—	—	—	2,870
88	—	—	—	—	2,592
89	—	—	—	—	2,353
90	—	—	—	—	2,139
91	—	—	—	—	1,951
92	—	—	—	—	1,775
93	—	—	—	—	1,604
94	—	—	—	—	1,436
95	—	—	—	—	1,274
96	—	—	—	—	1,097
97	—	—	—	—	905
98	—	—	—	—	691

Table 4-26.—Flood hydrograph development for subbasin 4—Continued.

Time, hours	Rain, inches	Loss, inches	Excess, inches	Unitgraph, ft^3/s	Hydrograph, ft^3/s
99	—	—	—	—	472
100	—	—	—	—	207
101	—	—	—	—	58
102	—	—	—	—	12
103	—	—	—	—	1
104	—	—	—	—	0

Chapter 5
FLOOD ROUTING THROUGH RESERVOIRS AND RIVER CHANNELS

5.1 General Considerations

It is common for the hydrologic engineer to encounter situations where a drainage basin is either so large or has such significant topographic variations that it must be subdivided into smaller subbasins for hydrologic analysis. In these situations, the component flood hydrographs for each subbasin must be routed and combined to arrive at the final flood hydrograph for the total drainage basin. Also, there are situations where there is one or more dams and their associated reservoirs located in the study basin. These facilities may have a regulating effect on the flood runoff prior to its reaching the point of interest. Any such regulating effects must be accounted for in determining the flood hydrograph at the point of interest. This chapter deals with the procedures used by the Bureau of Reclamation in routing these flood hydrographs through both reservoirs and channel reaches.

Consider the hypothetical drainage basin shown on figure 5-1, which has been subdivided into six subbasins for hydrologic analysis purposes. The total basin flood hydrograph at point E is required. An existing dam and reservoir have been assumed to be located at point B in subbasin 3. For purposes of this discussion, assume that concurrent flood hydrographs have been determined for each of the six subbasins. The routing and combining of these subbasin hydrographs to obtain the total hydrograph at point E would proceed as follows;

Step 1. The ordinates of the flood hydrographs for subbasins 1 and 2 at point A are added directly together to find the combined flood hydrograph at point A. This combined flood hydrograph is then channel routed to the reservoir at point B using one of the channel routing techniques described in section 5.2(a).

Step 2. The channel-routed hydrograph ordinates from point A are then added to the flood hydrograph ordinates from subbasin 3. This combined flood hydrograph represents the total inflow hydrograph to the reservoir at point B. This inflow hydrograph is then routed through the reservoir using the reservoir flood routing procedure described in section 5.2.

Step 3. The outflow hydrograph from the reservoir is then channel routed to point C.

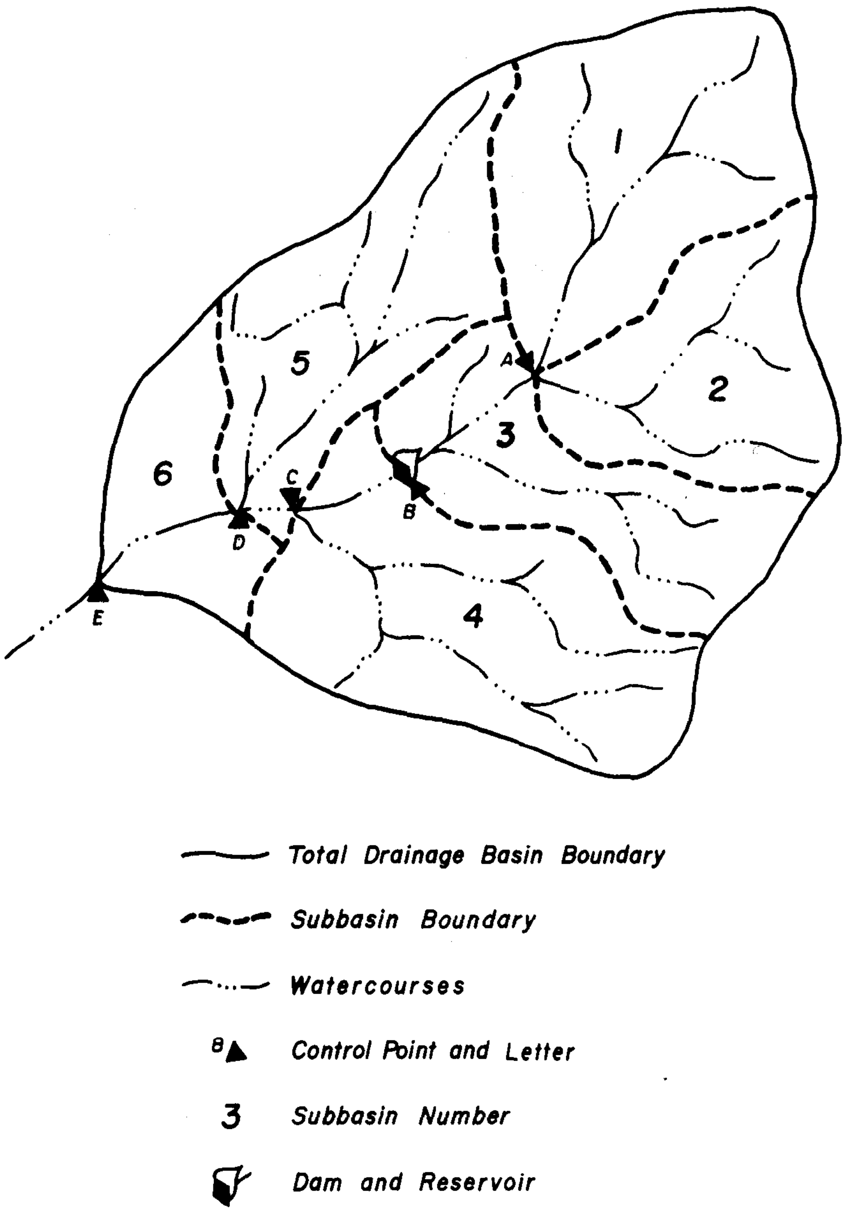


Figure 5-1.—Hypothetical drainage basin for flood hydrograph routing and combining considerations. 103-D-1919.

Step 4. The channel-routed flood hydrograph ordinates from the reservoir at point B are then added to the flood hydrograph ordinates from subbasin 4. This combined flood hydrograph represents the total flow at point C.

Step 5. The flood hydrograph representing the total flow at point C is then channel routed downstream to point D and added to the hydrograph for subbasin 5, which yields the total flood hydrograph at point D.

Step 6. The hydrograph from point D is then routed to point E and added to the flood hydrograph ordinates for subbasin 6. The resulting combined flood hydrograph represents the total flood hydrograph for the entire basin.

The previous example provides a general illustration of the type of flood routing problems encountered in most flood hydrology studies. The number of subbasins may be more or less than the six used in the example, depending on the individual hydrologic situation encountered. Also, the number of dams and reservoirs may be greater, or no such facilities may be present. While the general procedure is not complex, the "bookkeeping" associated with ensuring that all subbasins are added in the proper sequence and that hydrograph timing is in proper synchronization can become somewhat involved. To ensure that all routing and combining factors are properly accounted for, a table such as table 5-1 should be

Table 5-1.—Tabular procedure for routing and combining subbasin flood hydrographs to obtain total basin hydrograph.

Column No.	Procedure
(1)	Time, in hours
(2)	Subbasin 1 hydrograph ordinates at point A
(3)	Subbasin 2 hydrograph ordinates at point A
(4)	Combined hydrograph ordinates at point A, column (2) plus (3)
(5)	Column (4) ordinates routed to point B
(6)	Subbasin 3 hydrograph ordinates at point B
(7)	Combined hydrograph ordinates at point B, column (5) plus (6) Inflow to reservoir at point B
(8)	Outflow ordinates from reservoir at point B from reservoir routing
(9)	Column (8) ordinates routed to point C
(10)	Subbasin 4 hydrograph ordinates at point C
(11)	Combined hydrograph ordinates at point C, column (9) plus (10)
(12)	Column (11) ordinates routed to point D
(13)	Subbasin 5 hydrograph ordinates at point D
(14)	Combined hydrograph ordinates at point D, column (12) plus (13)
(15)	Column (14) hydrograph ordinates routed to point E
(16)	Subbasin 6 hydrograph ordinates at point E
(17)	Total basin hydrograph ordinates at point E, column (15) plus (16)

prepared for all studies when flood hydrograph routing and combining are part of the flood hydrology study. Table 5-1 reflects the physical condition of the basin as shown on figure 5-1. Each flood hydrology study should present a table similar to table 5-1 to facilitate full comprehension by anyone knowledgeable in the flood hydrology area.

5.2 Reservoir Flood Routings

The routing of flood hydrographs through reservoirs is considerably more straight forward and precise than routing through river channels. This is because the three necessary components of reservoir routing are known with some precision: (1) inflow hydrograph, (2) reservoir storage capacity verses elevation relationship, and (3) outflow rating curves for spillways and other release works. The storage accumulated or depleted in a reservoir is a function of the inflow and outflow rates. For a discrete interval of time Δt, the general relationship for routing flood hydrographs through reservoirs can be expressed by:

$$\Delta S = Q_i \,\Delta t - Q_o \,\Delta t \tag{1}$$

where:
ΔS = storage accumulated or lost during time interval Δt,
Q_i = average rate of inflow during time interval Δt, and
Q_o = average outflow during time interval Δt.

Referring to figure 5-2, the instantaneous rate of inflow at any time t can be determined from the hydrograph representing inflow to the reservoir. The rate of outflow from the reservoir is obtained from the rating curve that relates spillway discharge and reservoir water surface elevation, as shown on figure 5-3. Similar curves for the outlet works or other appurtenant release structures are used when studies indicate that these structures are reliable and will be available for accommodating the inflow flood. The change in storage is determined from the elevation capacity curve (fig. 5-4) that relates the reservoir water surface to its capacity at a given elevation.

The quantity of water a spillway can discharge depends on the size and type of spillway. For a simple overflow crest spillway, the discharge will vary with the hydraulic head on the spillway crest and the surcharge storage. However, in the case of a gated spillway, the outflow can be varied with respect to reservoir stage by operation of the gates. For example, one routing assumption for the operation of a gated spillway might be that the gates will be regulated so that inflow and outflow are equal until the gates are wide open; or an alternative assumption might be made that the gates will be opened at a known rate so that surcharge will accumulate before the gates are fully open.

Outflows during flood routings need not necessarily be limited to releases through spillways, but may be assumed to be supplemented by other

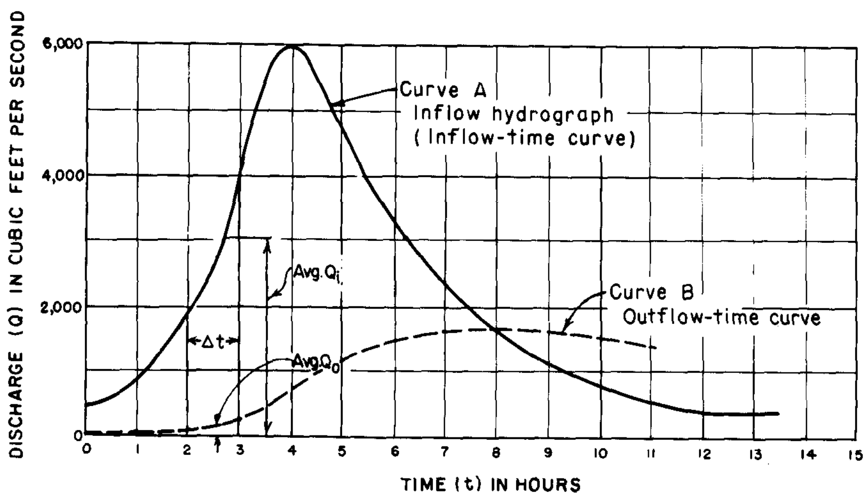


Figure 5-2.—Typical inflow and outflow hydrographs. 288-D-2399.

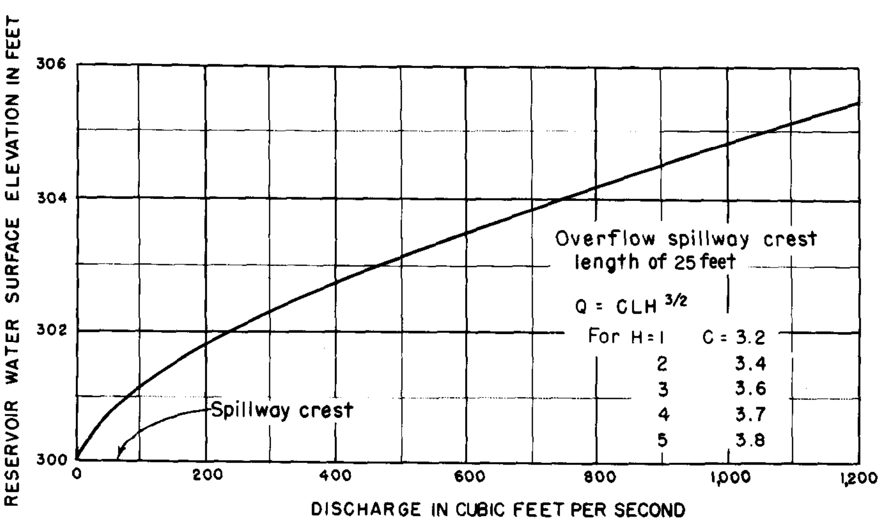


Figure 5-3.—Typical spillway discharge curve. 288-D-2401.

releases such as through river outlets, irrigation outlets, and powerplant turbines. In all such cases, the size, type, and method of operation of the spillway and other release works with reference to reservoir storage and flood inflow must be predetermined to establish an outflow versus elevation relationship for the reservoir routing. These operations should follow the standard operating procedures for existing reservoirs.

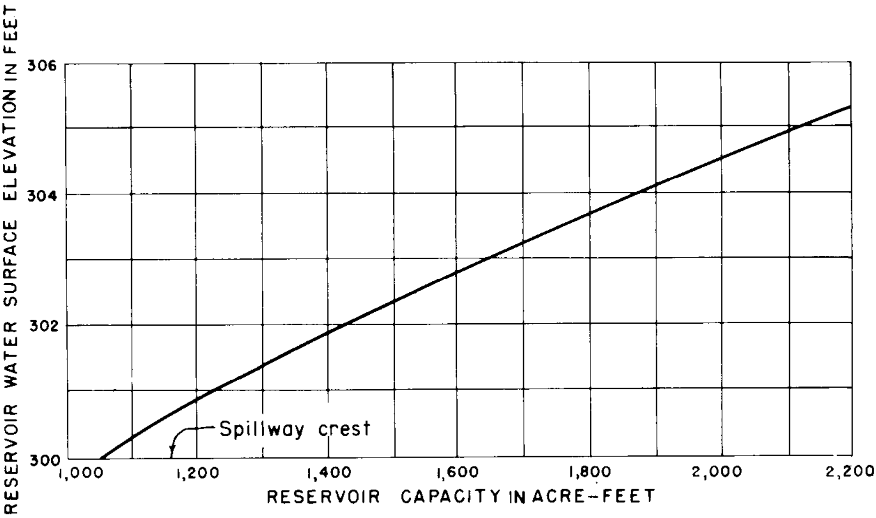


Figure 5-4.—Typical reservoir capacity curve. 288-D-2400.

If basic equations could be established for the inflow hydrograph curve, outflow releases (as may be modified by operational procedures), and the reservoir capacity curve, a solution of reservoir flood routing could be made by mathematical integration. However, general basic equations usually cannot be written for these variables, and such a solution is not possible. Many techniques for reservoir flood routing have been devised, each with advantages and disadvantages. These techniques vary from a strictly arithmetical method to an entirely graphical solution.

Electronic computers are routinely used to make reservoir flood routing computations. A computer program for reservoir flood routings is available from the Concrete Dams Branch at the Bureau's Denver Office. This computer program was developed using a mathematical (or linear) interpolation technique. For simplicity, a manual "trial and error" method similar to the computerized approach is used for illustration in this chapter. The data required for reservoir flood routing, which are the same regardless of the approach used, are as follows:

- Inflow hydrograph to reservoir, figure 5-2.
- Outflow versus elevation relationships, figure 5-3.
- Reservoir elevation versus capacity relationships, figure 5-4

The reservoir flood routing computations are shown in table 5-2, and the stepwise procedures for making these computations are:

Step 1. In column (1), list the time increments to be used in the computations. These increments are usually equal to the unit time period used in developing the reservoir inflow flood hydrograph discussed in section 4.1 (g)(1) of chapter 4. Longer increments may be used during the recession of the hydrograph; however, increments equal to the unit time period for the inflow hydrograph should always be used near the time the reservoir attains its maximum water surface level to ensure accurate computation of this critical elevation.

Step 2. List in column (2) the interval between time increments in column (1). These individual time increments will be referred to as Δt in the remaining steps.

Step 3. Column (3) is completed by reading the hydrograph ordinate in cubic feet per second at the time increment listed in column (1).

Step 4. Column (4) is the average inflow for time interval Δt in cubic feet per second. Values are obtained by averaging successive values in column (3).

Step 5. Values in column (5) are obtained by converting the values in column (4) from cubic feet per second to acre-feet. For these computations, 1 cubic foot per second flowing for 12 hours can be assumed to equal a volume of 1 acre-foot.

Step 6. Assume a trial water surface elevation in column (6), and then determine the corresponding rate of outflow from the outflow versus elevation relationship shown on figure 5-3; record this value in column (7).

Step 7. Average the rate of outflow determined in step 6 with the outflow for the reservoir water surface elevation that existed at the beginning of the time period; enter this average value in column (8).

Step 8. Obtain a value for column (9) by converting column (8) values in cubic feet per second for time interval Δt to acre-feet.

Step 9. Column (10) is column (5) minus column (9).

Step 10. The initial value in column (11) is the reservoir storage at the beginning of the inflow hydrograph to the reservoir. Determine subsequent values by adding the ΔS values from column (10) to the previous column (11) value.

Step 11. Use the reservoir capacity versus elevation relationship on figure 5-4 to determine the reservoir water surface elevation, and then enter in column (12) the reservoir surface elevation that corresponds to the storage shown in column (11).

Step 12. Compare the reservoir water surface elevation in column (12) with the trial reservoir elevation in column (6). If these elevations do not agree within a specified degree of accuracy, about 0.1 foot, make a second trial elevation in column (6) and repeat the procedure until the specified degree of accuracy is achieved.

The outflow versus time hydrograph that results from the reservoir flood routing shown in table 5-2 has been plotted as curve B on figure 5-2. Since the area under the reservoir inflow hydrograph (curve A) indicates the flood volume, the area under the outflow hydrograph (curve B) indicates the volume of outflow. Therefore, the volume indicated by the area between the two curves will be the surcharge storage in the reservoir during passage of the inflow flood. The surcharge storage computed for column (10) in table 5-2 can be checked by comparing with the measured area converted to acre-feet on figure 5-2.

5.3 Techniques for Routing Floods Through River Channels

The routing of floods through river channels may be accomplished using two basic approaches that are generally called the "Hydrologic Routing Approach" and the "Hydraulic Routing Approach." The hydrologic approach, for which there are many different application techniques, relies primarily on the basic storage relationship. Inflow to a river reach minus the outflow from that reach equals the change in storage volume contained in the reach. The hydraulic approach relies on consideration of the principles of conservation of both mass and energy.

Currently, there are four methods in general usage among the Bureau's various offices: (1) Successive Average Lag Method, (2) Modified Puls Method, (3) Modified Wilson Method, and (4) Muskingum Method. Of the four techniques, only the Modified Puls Method is strictly a hydrologic routing approach, the others fall somewhere in between the two approaches. These techniques are described in the following sections.

5.4 Successive Average Lag Method

This method, developed by Fred E. Tatum [62],[1] is not as sophisticated as alternative methods that more rigorously account for the effect of channel storage on the attenuation of a flood hydrograph. However, this method does provide reasonable results for project designs when the flood wave travel times are based on the analysis of actual major floods in the basin of interest or in nearby hydrologically and hydraulically similar basins. Use of this method is suitable for preauthorization type planning studies. If suitable data are available, as discussed later, methods using "storage routing" techniques should be used for studies supporting postauthorization or final designs. The following discussion relative to the basis of Tatum's method and procedures for its application has been substantially extracted from Corps of Engineers Manual *Routing of Floods*

[1]Numbers in brackets refer to entries in the Bibliography.

Table 5-2.—Flood routing computations.

(1) Time t, hours	(2) Δt, hours	(3) Inflow at time t, ft³/s	(4) Average rate of inflow, Q_i for Δt, ft³/s	(5) Inflow, acre-feet	(6) Trial reservoir storage elevation at time t	(7) Outflow at time t, ft³/s	(8) Average rate of outflow, Q_o, for Δt, ft³/s	(9) Outflow, acre-feet	(10) Incremental storage ΔS, acre-feet	(11) Total storage, acre-feet	(12) Reservoir elevation at end of Δt, feet	(13) Remarks
0		400				0				1,050	300.3	
	1		600	50	~~300.2~~	~~5~~	~~3~~	~~0~~	~~50~~	~~1,100~~	~~300.3~~	~~High~~
1		800			300.3	10	5	0	50	1,100	300.3	OK
	1		1,400	117	~~300.8~~	~~84~~	~~32~~	~~3~~	~~114~~	~~1,214~~	~~301.0~~	~~High~~
2		2,000			301.0	80	45	4	113	1,213	301.0	OK
	1		3,000	250	~~302.3~~	~~300~~	~~190~~	~~16~~	~~234~~	~~1,447~~	~~302.1~~	~~Low~~
3		4,000			302.1	260	170	14	236	1,449	302.1	OK
	1		5,000	417	~~303.9~~	~~710~~	~~485~~	~~40~~	~~377~~	~~1,826~~	~~303.8~~	~~Low~~
4		6,000			303.8	690	475	40	377	1,826	303.8	OK
	1		5,350	446	~~305.6~~	~~1,060~~	~~675~~	~~73~~	~~373~~	~~2,199~~	~~305.3~~	~~High~~
5		4,700			305.3	1,160	925	77	369	2,195	305.3	OK
	1		4,000	333	~~306.3~~	~~1,500~~	~~1,900~~	~~111~~	~~222~~	~~2,417~~	~~306.2~~	~~Low~~
6		3,300			306.2	1,470	1,315	110	223	2,418	306.2	OK
	1		2,850	238	306.6	1,610	1,540	128	110	2,528	306.6	OK
7		2,400										
	1		2,000	167	306.7	1,650	1,630	136	31	2,559	306.7	OK
8		1,600										
	1		1,350	112	306.6	1,610	1,630	136	−24	2,535	306.6	OK
9		1,100										
	2		800	133	306.0	1,400	1,505	251	−118	2,417	306.1	High
11		500			306.1	1,430	1,520	253	−120	2,415	306.1	OK

Through River Channels [62]. The basic premises of the Successive Average Lag Method are as follows:

- The storage versus flow relationship and the associated flood hydrograph shape tend to vary uniformly along a watercourse and its associated flood plain. This is presumed due to the floodway having been configured by successive inflows from the tributary area over a considerable period.

- The shape of the observed hydrograph reflects the cumulative effect of the storage conditions of the drainage network above the measuring point.

- If I_1 and I_2 represent flood hydrograph ordinates at times t_1 and t_2 at measuring station A then, at a downstream measuring station B, the inflow at time t_2 is equal to the mean inflow $(I_1 + I_2)/2$ at station A for the time interval t_1 to t_2. It is assumed that this relationship between inflows at stations A and B applies to all time periods with the same time interval as t_1 to t_2.

- It is assumed that the hydrograph at B reflects, in its altered shape, the change due to storage conditions in the reach between A and B. Therefore, the process may be repeated for as many subreaches as desired to determine the change in shape of the hydrograph as a result of routing through channel storage. It is likely that the successive hydrographs obtained by this procedure are not necessarily spaced at equal intervals along the stream, but that they are equally spaced in time. For some subreaches, the velocity of translation of the flood wave may be more or less than for others due to local variations in storage conditions.

(a) Basis of Method.—Further consideration of the method discloses its properties. On figure 5-5, let hydrograph A be defined by inflow discharges $I_0, I_1, I_2, \ldots I_n$ to the routing reach at times $t_0, t_1, t_2, \ldots t_n$, with time intervals Δt small enough so that the inflow discharge varies linearly. Hydrograph B is hydrograph A translated without change in shape to a downstream point to which the travel time is $t/2$. The inflow discharges at times $t_1, t_2, \ldots t_{n+1}$ are :

$$(I_0 + I_1)/2, (I_1 + I_2)/2 \ldots (I_{n-1} + I_n)/2$$

By drawing straight lines between these inflow discharge hydrograph the routed reach outflow discharge for hydrograph C is obtained, which represents the "first-step" hydrograph of the successive average lag method. In each successive subreach, the midpoint inflow discharges of

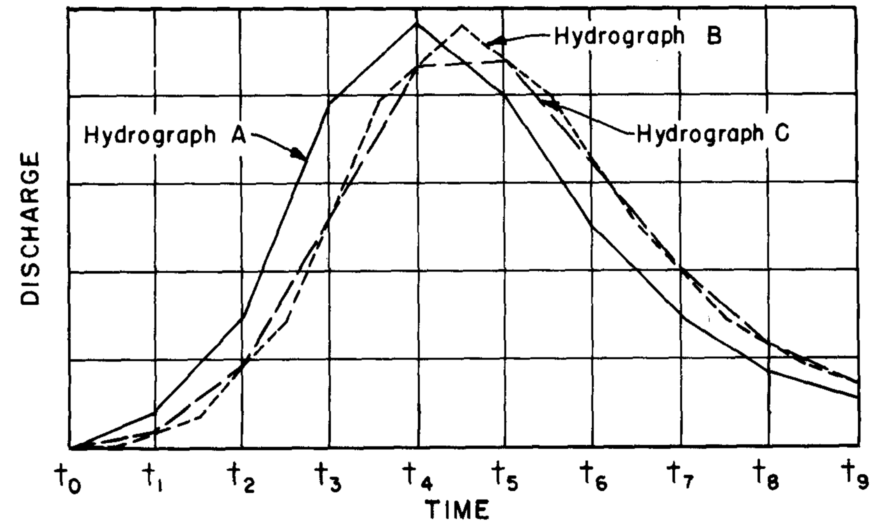


Figure 5-5.—Hypothetical flood hydrograph routed by Tatum Method [62]. 103-D-1920.

the preceding hydrographs are connected, which results in a flattening of the wave form in its downstream procession. Consideration of the geometrical properties inherent in this method is sufficient to disclose that the crest and shape of the hydrograph, at some distant downstream point, varies with the choice of the time increment of the successive steps. That is, the smaller the time increment, the more the routed hydrograph retains the shape of the original hydrograph.

(b) Routing Constants and Routing Procedure.—The ordinates of a hydrograph routed successively through n subreaches may be expressed in terms of the outflow ordinates, O, of the original hydrograph. Thus, if:

$$O_3 = \frac{1}{2}\left[\frac{I_1 + I_2}{2} + \frac{I_2 + I_3}{2}\right] = \frac{I_1 + 2I_2 + I_3}{4} \qquad (2)$$

and, if there are three subreaches:

$$O_4 = \frac{I_1 + 3I_2 + 3I_3 + I_4}{8} \qquad (3)$$

If the inflow ordinates are I_1, I_2, I_3. . . . .I_{n+1}, and n is the number of subreaches,

$$O_{n+1} = c_1 I_1 + c_2 I_2 + c_3 I_3 + \ldots + c_{n+1} I_{n+1} \qquad (4)$$

where:

$$c_1 = \frac{1}{2^n}$$

$$c_2 = \frac{n}{2^n}$$

$$c_3 = \frac{n(n-1)}{2^n 2\,!}$$

$$c_4 = \frac{n(n-1)(n-2)}{2^n 3\,!}$$

$$c_n = \frac{n(n-1)(n-2)\ldots.(2)}{2^n (n-1)\,!}$$

$$c_{n+1} = \frac{n!}{2^n n\,!} + \frac{1}{2^n}$$

Routing constants are shown in table 5-3 for the various numbers of routing subreaches.

The number of routing steps for a particular application can be determined by dividing the travel time in hours (from the point where the hydrograph is known to the point of interest) by the unit time of the hydrograph, and then multiplying the result by 2. The unit time of the hydrograph is usually the same as for the unit hydrograph used to develop the hydrograph. The factors associated with the appropriate number of routing steps are obtained from table 5-3. For a particular application, the factors are applied to the known hydrograph in the same manner as the rainfall excess was applied to the unit hydrograph to obtain the runoff hydrograph, with the routing constants being similar to the precipitation excess increments and the known hydrograph assumed to be the unit hydrograph. It should be noted that the sum of the routing constants for any routing step is always 1.000.

5.5 Modified Puls Method

The original Puls Method developed by L. G. Puls [69] was also known as the Method of Inflow-Storage-Discharge Curves. The modified method is similar to the original except that the routing process is simplified by using only one curve, which is designated the "storage-indication curve" (outflow versus $S + O/2$), in the routing process. This curve is also applicable to the routing of floods through reservoirs.

The storage-indication curve is drawn using instantaneous outflows, which are obtained from rating curve at lower end of reach, as ordinates and corresponding storage plus one-half of outflow ($S + O/2$) values as

Table 5-3.—Flood routing constants by Successive Average Lag Method.

	Number of Routing Steps									
	1	2	3	4	5	6	7	8	9	10
c_1	0.5000	0.2500	0.1250	0.0625	0.0313	0.0156	0.0078	0.0039	0.0020	0.0010
c_2	.5000	.5000	.3750	.2500	.1562	.0937	.0547	.0313	.0176	.0098
c_3		.2500	.3750	.3750	.3125	.2344	.1641	.1094	.0703	.0440
c_4			.1250	.2500	.3125	.3126	.2743	.2187	.1741	.1172
c_5				.0625	.1562	.2344	.2734	.2734	.2560	.2050
c_6					.0313	.0937	.1641	.2187	.2460	.2460
c_7						.0156	.0547	.1095	.1641	.2050
c_8							.0078	.0313	.0703	.1172
c_9								.0039	.0176	.0440
c_{10}									.0020	.0098
c_{11}										.0010

abscissas. The significance of this curve in routing can be readily understood by referring to the following basic equation, which is a rearrangement of the traditional inflow minus outflow equals change in storage equation:

$$\left(\frac{I_1 + I_2}{2}\right)\Delta t + \left(S_1 - \frac{O_1}{2}\right)\Delta t = \left(S_2 + \frac{O_2}{2}\right)\Delta t \tag{5}$$

where:
- I_1 = inflow at start of time period t, in cubic feet per second;
- I_2 = inflow at end of time period t, in cubic feet per second;
- O_1 = outflow at lower end of reach at start of time period t, in cubic feet per second;
- O_2 = outflow at downstream end of reach at end of time period t, in cubic feet per second;
- S_1 = storage in reach at start of time period t, in cubic feet; and
- S_2 = storage in reach at end of time period t, in cubic feet.

Note that the right side of equation (5) corresponds to the term on the abscissa scale of figure 5-6. Also, by subtracting O_2 (ordinate scale of figure 5-6) from $S_2 + O_2/2$, or $(S_2 + O_2/2) - O_2$, yields $S_2 - O_2/2$. This resulting expression is identical to the $S_1 - O_1/2$ on the left side of

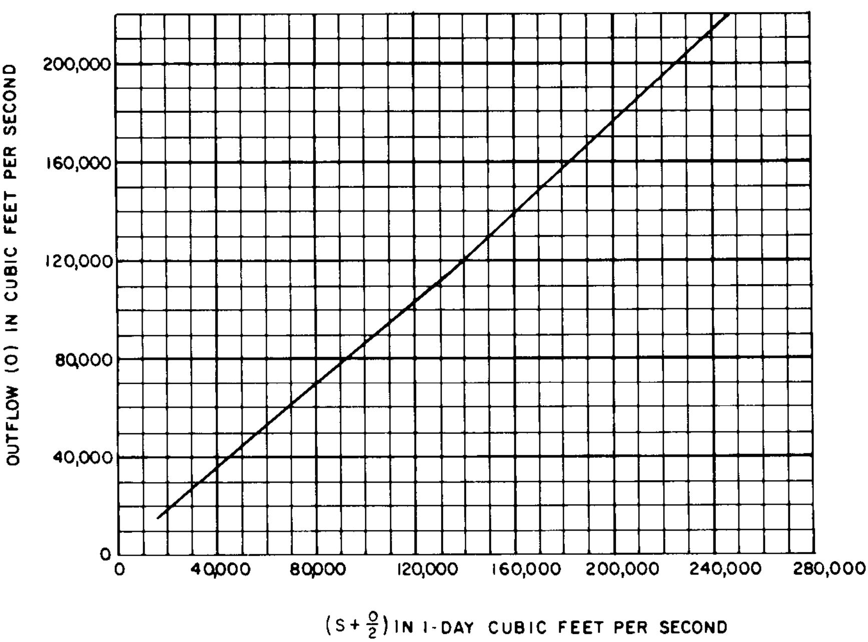


Figure 5-6.—Storage indication curve. 103-D-1921.

equation (5) except for the subscripts. Since subscript 1 denotes values at the start of a time increment and subscript 2 denotes values at the end of a time increment, then $S_2 - O_2/2$ at the end of one time increment is numerically equal to $S_1 - O_1/2$ at the start of the succeeding time increment. The detailed routing process is as follows:

Step 1. Compute a numerical value for the left side of equation (5) by substituting given values of I_1, I_2, S_1, and O_1 for the first time increment.

Step 2. Knowing the left side of equation (5), which is equal to $S_2 + O_2/2$, enter the storage-indication curve (fig. 5-6) and read the O_2 outflow corresponding to the computed value for $S_2 + O_2/2$. This outflow is the instantaneous outflow from the reach at the end of the first time increment.

Step 3. Subtract the O_2 outflow obtained from figure 5-6 from the corresponding computed abscissa, $S_2 + O_2/2$, which gives a numerical value for $S_2 - O_2/2$.

The value of $S_1 - O_1/2$ for the second time increment is equal to $S_2 - O_2/2$ for the first time increment. Consequently, the left side of equation (5) can be computed for the second time increment and the procedure repeated.

A tabular example of the Modified Puls Method for routing computations is shown in table 5-4, where the flood hydrograph shown on figure 5-7 is routed over a single river reach. The storage-indication curve representative of that reach is shown on figure 5-6. The given O_2 outflow from the river reach of 70,000 cubic feet per second at the start of first time increment is shown in column (5) of table 5-4. Given this outflow, a value for $S_2 + O_2/2$ in equation (5) can be found from figure 5-6, and is found to be 82,000 1-day cubic feet per second. Subtracting the outflow of 70,000 from 82,000 yields a value for $S_2 - O_2/2$ of 12,000 1-day cubic feet per second. Solving for the left side of equation (5):

$$\frac{I_1 + I_2}{2} + \left(S_1 - \frac{O_1}{2}\right) \text{ or } \frac{70{,}000 + 130{,}000}{2} + 12{,}000 = 112{,}000 \text{ 1-day ft}^3/\text{s}$$

The outflow from the river reach at the end of the first day may then be determined from figure 5-6 using an $S_2 + O_2/2$ value of 112,000 and reading an outflow value of 95,000 on the ordinate scale. To determine outflow at end of second day, the value of $S_2 - O_2/2$ is determined by subtracting the outflow at end of first day from corresponding $S_2 + O_2/2$ value, or 112,000 − 95,000 = 17,000 1-day cubic feet per second. Again, solve for left side of equation (5):

$$\frac{I_1 + I_2}{2} + \left(S_1 - \frac{O_1}{2}\right) \text{ or } \frac{130{,}000+240{,}000}{2} + 17{,}000 = 202{,}000 \text{ 1-day ft}^3/\text{s}$$

Table 5-4.—Example on routing computations by Modified Puls Method.

(1) Time, days	(2) Instantaneous Inflow I, ft³/s	(3) Average Inflow per Day $\frac{I_1 + I_2}{2}$, ft³/s	(4) Outflow $S + \frac{O}{2}$, 1-day ft³/s	(5) Computed Outflow, ft³/s
0	70,000		82,000	70,000*
		100,000		
1	130,000		112,000	95,000
		185,000		
2	240,000		202,000	176,000
		211,000		
3	182,000		237,000	209,000
		160,000		
4	138,000		188,000	163,000
		121,500		
5	105,000		146,500	125,000
		90,500		
6	76,000		112,000	95,000
		63,000		
7	50,000		80,000	68,000

*This value is either known or assumed for start of first time increment in any routing problem.

The outflow at end of second day can then be determined using figure 5-6, where the outflow value corresponding to an $S_2 + O_2/2$ value of 202,000 1-day cubic feet per second is 176,000 cubic feet per second. Table 5-4 shows that this process is repeated for each time increment until entire flood hydrograph has been routed through the reach.

In the preceding example, which illustrates the basic principles of the Modified Puls Method, gauging stations were in operation at both ends of the reach, which facilitated the derivation of the storage and discharge relationships. Frequently, gauging stations will not be available and these relationships will have to be computed using Manning's Equation:

$$V = \frac{1.486\ R^{2/3}\ S^{1/2}}{n} \tag{6}$$

where:

V = velocity, in feet per second;
R = hydraulic radius, in feet, which is found by dividing cross-sectional area of flow by wetted perimeter of channel;
S = slope of channel, in feet per feet; and
n = Manning's hydraulic roughness coefficient.

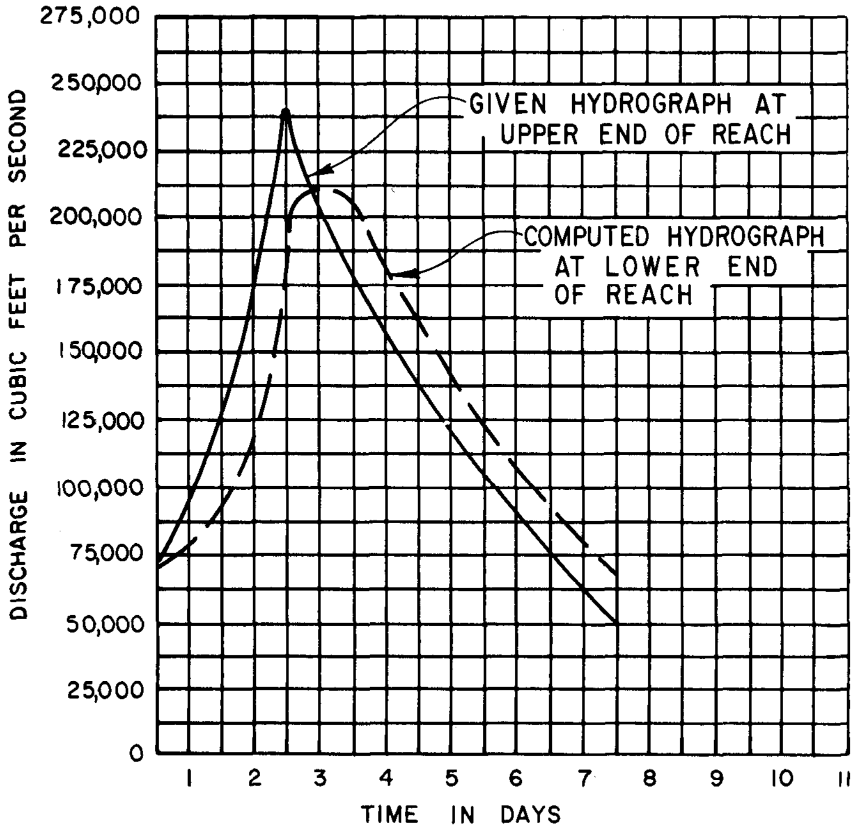


Figure 5-7.—Hypothetical flood hydrograph routed by Modified Puls Method. 103-D-1922.

The discharge Q, in cubic feet per second, can then be found by multiplying the velocity V, in feet per second, times the cross-sectional area A, in square feet, or $Q = VA$. The basic data required to apply equation (6) are slope of streambed, typical cross section for reach, and a coefficient of roughness for the reach. Using equation (6), the average flow velocity can be determined for various depths of flow and, from the flow velocities and average cross-section, the outflow in cubic feet per second at the lower end of the reach can be computed for various depths of water. Also, the storage in the reach can be computed for various depths of water by calculating the volume in the reach corresponding to the depths of water on the average cross section. With the discharge at the lower end of the reach and the volume in the reach both related to a common factor, depth of flow, the storage-indication curve (fig. 5-6) can be derived.

In the preceding example, it was assumed that all of the water that entered the upper end of the reach was discharged at the lower end of the reach. In many cases, this is not exactly true because some of the water will infiltrate by percolating into the streambed as the water flows downstream. This percolation loss will vary with the geologic and soil conditions of the flood plain but, in most cases, the loss will be less than 3 cubic feet per second per wetted acre of streambed overbank area. Percolation losses may be reduced to zero, or actually become negative, if the ground-water inflow is appreciable. However, for illustration purposes, a percolation loss will be assumed in the next example. By applying the wetted perimeter for various depths of water, the wetted acres in the reach can be computed. Having determined or assumed a percolation loss in cubic feet per second per wetted acre, a curve for percolation loss versus outflow can be drawn.

In the previous example, it was assumed that the total outflow at the lower end of the reach originated at or above the upper end of the reach with no contribution to flow from the intervening tributaries. This condition simplifies the routing process, but frequently the tributary inflow in the reach is relatively large and should be considered in the routing process. The storage-discharge relationship derived in previous examples is for only one given condition, that of no tributary inflow.

The following example is given to illustrate the Modified Puls Method for a stream that has no gauging stations in the reach but has a percolation loss. Tributary inflow is not considered in this example as it will be discussed in subsequent sections. The following basic data are given:

Length of reach = 10.7 miles
Change in elevation of streambed over length of reach = 800 feet
Coefficient of roughness, n = 0.075
Typical cross section shown on table 5-5
Percolation loss assumed to be 0.5 cubic feet per second per wetted acre

The computations used to derive the storage-indication curve and the percolation loss versus outflow curve are shown in table 5-5. These two curves, shown on figure 5-8, are determined by plotting the column (7) values in table 5-5 versus corresponding values of column (9), and column (7) versus column (11), respectively. After the storage-indication curve is derived, the procedure is identical to that described in the previous example. The results of the routing computations are shown in table 5-6, which is comparable to table 5-4 used in the previous example except that the percolation loss is included in table 5-6. The given hydrograph at the upper end of the reach and the computed hydrograph at the lower end of the reach are shown on figure 5-9.

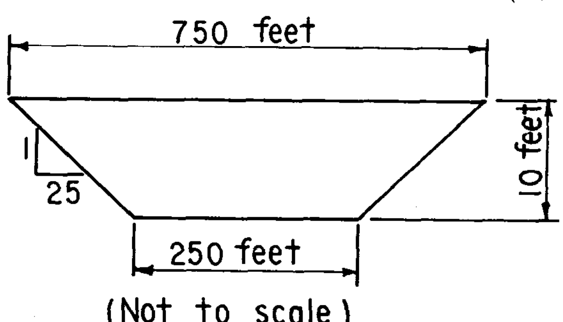
5.6 Modified Wilson Method

This method is based on a modification of Walter T. Wilson's streamflow routing procedure [63]. This method recognizes both the translation

Table 5-5.—Stream routing data by Modified Puls Method.

(1) Depth, feet	(2) Wetted Perimeter, feet	(3) Area A, ft²	(4) Hydraulic Radius R, feet	(5) $R^{2/3}$	(6) Average Water Velocity V, ft/s	(7) Discharge Q (3) × (6), ft³/s	(8) Storage* S in Reach, 1-hour ft³/s	(9) $S + \frac{Q}{2}$, 1-hour ft³/s	(10) Wetted Area, acres	(11) Percolation Loss ft³/s
1	300	275	0.92	0.95	2.24		4,318		389	194
2	350	600	1.71	1.43	3.37	2,022	9,420	10,431	454	227
3	400	975	2.44	1.81	4.27	4,163	15,308	17,389	519	259
4	450	1,400	3.11	2.13	5.02	7,028	21,980	25,494	583	291
5	500	1,875	3.75	2.41	5.68	10,650	29,438	34,763	648	324
6	550	2,400	4.36	2.67	6.30	15,120	37,680	45,240	713	356
7	600	2,975	4.96	2.91	6.86	20,409	46,708	56,912	778	389
8	650	3,600	5.54	3.13	7.38	26,568	56,520	69,804	843	421
9	700	4,275	6.11	3.34	7.88	33,687	67,118	83,962	908	454
10	750	5,000	6.37	3.54	8.35	41,750	78,500	99,375	973	485

*Storage in 1-hour cubic feet per second: S = Area times $\frac{(56{,}496)\,(24)}{(43{,}560)\,(1.983)}$ = Area times 15.70



Length of reach = 10.7 miles = 56,496 feet

Slope = 800/56,496 = 0.01416, n = 0.075

$$R = \frac{\text{Cross-sectional area}}{\text{Wetted perimeter}}$$

$$Q = AV$$

$$V = \frac{1.486\,R^{2/3}\,S^{1/2}}{n} = \frac{1.486\,R^{2/3}\,(0.01416)^{1/2}}{0.075} = 2.358\,R^{2/3}$$

Percolation loss = 0.5 cubic feet per second per wetted acre of channel.

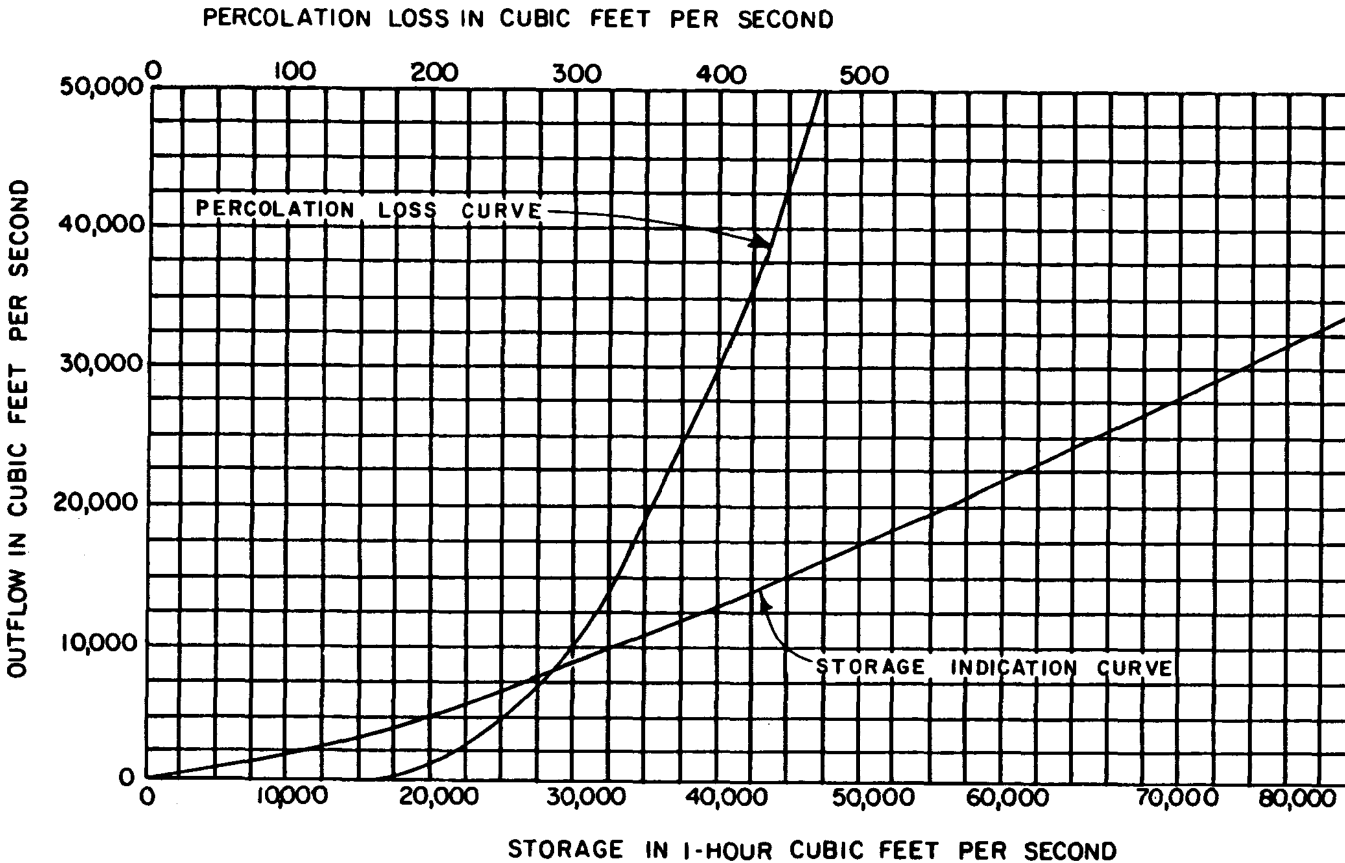


Figure 5-8.—Storage indication and percolation loss curves. 103-D-1923.

Table 5-6.—Stream routing computations by Modified Puls Method.

(1) Time Period, hours	(2) Average Inflow Per Hour, ft³/s	(3) Storage Indication $\left(S + \frac{Q}{2}\right)$ 1-hour ft³/s	(4) Instantaneous Outflow, ft³/s	(5) Instantaneous Outflow Minus Percolation, ft³/s
0 - 1	1,000	6,000	1,000	800
2	1,000	6,000	1,000	800
3	1,250	6,250	1,000	800
4	1,750	7,000	1,200	1,000
5	2,250	8,050	1,400	1,200
6	3,250	9,900	1,900	1,700
7	4,500	12,500	2,700	2,500
8	7,250	17,050	4,100	3,900
9	11,750	24,700	6,800	6,500
10	19,750	37,650	11,900	11,600
11	26,500	52,250	18,300	18,000
12	29,750	63,700	23,700	23,300
13	28,000	68,000	25,800	25,400
14	25,500	67,700	25,700	25,300
15	22,750	64,750	24,100	23,700
16	20,250	60,900	22,100	21,700
17	17,500	56,300	20,200	19,800
18	14,700	50,800	17,700	17,400
19	12,750	45,850	15,500	15,200
20	11,250	41,600	13,600	13,300
21	10,000	38,000	12,100	11,800
22	9,000	34,900	10,900	10,600
23	8,250	32,250	9,800	9,500
24	7,500	29,950	8,800	8,500
25	6,800	27,950	8,000	7,700
26	6,250	26,200	7,300	7,000
27	5,750	24,650	6,700	6,400
28	5,300	23,250	6,200	5,900
29	4,800	21,850	5,700	5,400
30	4,500	20,650	5,300	5,000
31	4,200	19,550	4,900	4,600
32	3,800	18,450	4,600	4,400
33	3,600	17,450	4,200	4,000
34	3,300	16,550	3,900	3,700
35	3,100	15,750	3,600	3,400
36	2,900	15,050	3,400	3,200
37	2,800	14,450	3,200	3,000
38	2,750	14,000	3,100	2,900
39	2,650	13,550	3,000	2,800
40	2,550	13,100	2,900	2,700
41	2,450	12,650	2,800	2,600
42	2,400	12,250	2,600	2,400
43	2,350	12,000	2,500	2,300
44	2,300	11,800	2,400	2,200
45	2,250	11,650	2,300	2,100
46	2,200	11,550	2,300	2,100
47	2,150	11,400	2,200	2,000
48	2,100	11,300	2,200	2,000

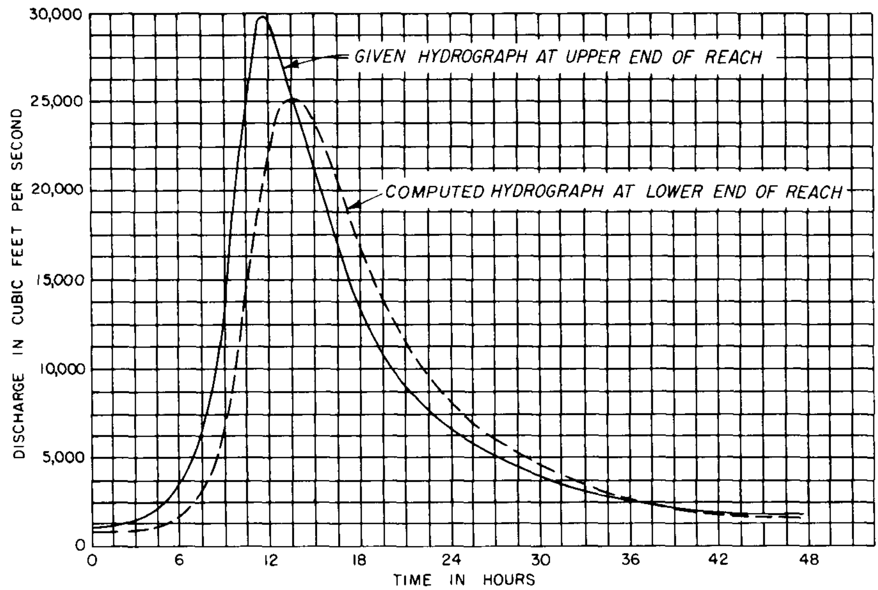


Figure 5-9.—Example of a routed flood hydrograph by Modified Puls Method. 103-D-1924.

time of a flood wave and the effects of channel storage on peak discharge attenuation. The basic mathematical equation used in this method has as its basis the standard relationship, which is inflow minus outflow equals change in storage over some discrete time interval. Based on this fundamental relationship, the Modified Wilson Equation takes the form:

$$O_2 = O_1 + K(I_1 + I_2 - 2O_1) \tag{7}$$

where:

O_1 and O_2 = consecutive incremental outflow discharges at downstream limit of stream reach,

I_1 and I_2 = consecutive incremental inflow discharges at upstream end of stream reach, and

K = a coefficient dependent on translation time of flood wave T_r, channel storage component (expressed as a function of time T_s, and the routing time interval Δt.

The value of K is then found by applying the equation:

$$K = \frac{\Delta t}{2T + \Delta t} \text{ for } T < 0.5\ T_s \tag{8}$$

where:

T = travel time, in hours of peak flow through reach, and consists of the two components T_r and T_s.

Investigators have found that this method is relatively well verified in the mountainous areas of California. In these areas, the translation time T_r and storage time T_s are about equal. This method has also been used successfully in the Texas high plains basins. In either mountainous or plains basins, it is imperative that the travel time value T be determined from the analysis of an observed event on a hydraulically similar channel with respect to slope, channel cross section, and hydraulic roughness characteristics.

5.7 Muskingum Routing Method

This method is one of the more sophisticated of the hydrologic flood routing procedures. The Muskingum or Modified Puls method should be used in flood hydrology studies to support final designs leading to project construction. The following explanation of the Muskingum Method has been substantially extracted from the Corps of Engineers *Routing of Floods Through River Channels* [62]. This method, sometimes called the "Coefficient Method of Routing," was developed for application by COE in connection with studies of the Muskingum Conservancy District Flood Control Project in the mid 1930's.

The Muskingum method stems from an assumption of relating storage within a routing reach to outflows at each end of the reach, which results in the basic equation:

$$S = K\,[XI + O\,(1 - X)] = KO + KX\,(I - O) \qquad (9)$$

where:

S = storage in the reach,
K and X = constants, and
I and O = simultaneous inflow and outflow, respectively, for the reach.

The term KO in equation (9) has been considered as representing the prism storage under the profile for steady flow O (fig. 5-10), and the term $KX\,(I - O)$ as representing the wedge storage produced by variations from a steady-flow profile due to differences between inflow and outflow occurring during rising and falling stages. For application of flood routing, equation (9) is expressed in increments and combined with the continuity equation (1) in section 5.2:

$$\Delta S = 0.5\,\Delta t\,[I_1 + I_2) - (O_1 + O_2)] \qquad (10)$$

to yield:

$$O_2 - O_1 = C_1\,(I_1 - O_1) + C_2\,(I_2 - I_1) \qquad (11)$$

where the subscripts refer to the beginning and ending of time period t, and coefficients C_1 and C_2 have values of:

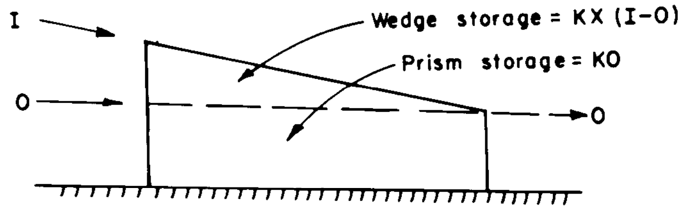


Figure 5-10.—Muskingum routing principles. 103-D-1925.

$$C_1 = \frac{2\Delta t}{2K(1 - X) + \Delta t} \text{ and } C_2 = \frac{\Delta t - 2KX}{2K(1 - X) + \Delta t}$$

Routing of a hydrograph using equation (11) is not an involved process. For the example shown in table 5-7, an inflow hydrograph for a reach of the Tuscarawas River in the Muskingum Basin is considered to be represented adequately by instantaneous values of inflow at half-day intervals. The inflows are tabulated in column (2) of table 5-7. In this example, it is assumed there is no tributary inflow and no loss in the reach. Also given are the conditions that $K = \Delta t = 0.5$ day, and $X = 0.3$. From equation (11), the values of C_1 and C_2 are computed, then C_2 is multiplied in turn by each increment of inflow to obtain column (4). Therefore, the first entry in column (4) is $C_2 \ (I_2 - I_1) = 1/6 \ (7.0 - 2.0) = 0.8$. The first entry for outflow in column (5) is assumed to be equal to the inflow at the corresponding time. The first entry in column (3) is $C_1 \ (I_1 - O_1) = (1/1.2) \ (2.0 - 2.0) = 0$. The second entry in column (5) is the summation of columns (3), (4), and (5) of the preceding line. The entries in columns (3) and (5) are determined alternately until column (5) is completed, which also completes the routing.

The use of constant values for coefficients K and X throughout a flood routing is not adequate in all instances; for example, in operating problems requiring accurate stage forecasts throughout a flood period. The coefficient method may be expanded by using a variable for K and/or X. If these coefficients are assumed to vary as functions of outflow, the routing may be made with several values, each applicable to a specific range of outflow discharges; or working curves using C_1 and C_2 may be plotted against the outflow for use as the routing proceeds step by step. When using varying values of K and X, the computed outflow hydrograph may be found to have more or less volume than the inflow hydrograph. A readjustment of the relationship between the coefficients and outflow or stage is one means of bringing the volumes into balance.

The practical consideration in selecting Δt, length of routing period, is that its value is small enough to define the hydrographs. A secondary consideration is that the water-surface profile in the reach will be relatively straight. The relationship of Δt to the constant K is discussed in

Table 5-7.—Flood routing by Muskingum Method. Tuscarawas River, Muskingum Basin, Ohio Reach from Newcomerstown to Coshocton, February 26 to March 4, 1929.

For t = 0.5 day, K = 0.5 day, and X = 0.3:

$$C_1 = \frac{2\,\Delta t}{2K(1-X) + \Delta t} = \frac{1}{1.2} \quad \text{and } C_2 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} = \frac{1}{6}$$

(1) Date	(2) Inflow I (Newcomerstown) ft³/s	(3) $C_1(I_1-O_1)$ $C_1 = 1/1.2$ ft³/s	(4) $C_2(I_2-I_1)$ $C_2 = 1/6$ ft³/s	(5) Outflow O (Coshocton) ft³/s
2-26-29 a.m.	2,000	0	800	2,000
p.m.	7,000	3,500	800	2,800
2-27-29 a.m.	11,700	3,800	800	7,100
p.m.	16,500	4,000	1,200	11,700
2-28-29 a.m.	24,000	5,900	800	16,900
p.m.	29,100	4,600	−100	23,600
3-01-29 a.m.	28,400	200	−800	28,100
p.m.	23,800	−3,100	−700	27,500
3-02-29 a.m.	19,400	−3,600	−700	23,700
p.m.	15,300	−3,400	−700	19,400
3-03-29 a.m.	11,200	−3,400	−500	15,300
p.m.	8,200	−2,700	−300	11,400
3-04-29 a.m.	6,400	−1,700	−200	8,400
p.m.	5,200	—	—	6,500

section 5.7 (a), which also covers the determination of K and X and the selection of reach lengths. A routed hydrograph is relatively insensitive to small changes in Δt when X and K are held constant.

(a) Determination of* K *Constant.—To show the significance of the constant K in equation (9), figure 5-11 shows an application of this storage equation to looped storage curves derived from flood hydrographs for a river reach. Figures 5-11(a) and 5-11(b) differ in the ordinate scale, with the objective of showing that, by a suitable selection of X, the storage loop of figure 5-11 (a) may be reduced to the relatively small loops shown on figure 5-11 (b). Or these two figures, the value of K for an incremental time period is the reciprocal of the slope of the mean line representing storage versus outflow; that is, K is the change of storage per unit change of discharge.

As defined above, K has the dimension of time, and it can be shown that the time interval represented by K is equivalent to the time required for an elemental discharge wave to traverse the routing reach. This indicates that a relationship should exist between K and Δt, the selected length of routing period. For example, if Δt is less than K, an elemental discharge wave will not have traversed the reach during a routing period and the computed hydrograph for the downstream end of the reach, which is based in part on changes in the discharge at the upstream end of the reach during the routing period, cannot represent the actual hydrograph. From such reasoning, it would appear that Δt should equal K. In actual applications, a moderate departure from this equality is permissible because the routed hydrograph is relatively insensitive to the value of Δt. However, Δt should not be less than $2KX$ to avoid negative values of C_2.

The relationship between K and the celerity of the elementary wave also signifies that K is very close to the time interval between the centers of mass of the inflow and outflow hydrographs.

From the previous discussions, it is obvious that K can be determined by four different methods:

(1) The K value for a reach between two stations for which flood hydrographs are available may be taken as the time of travel of the

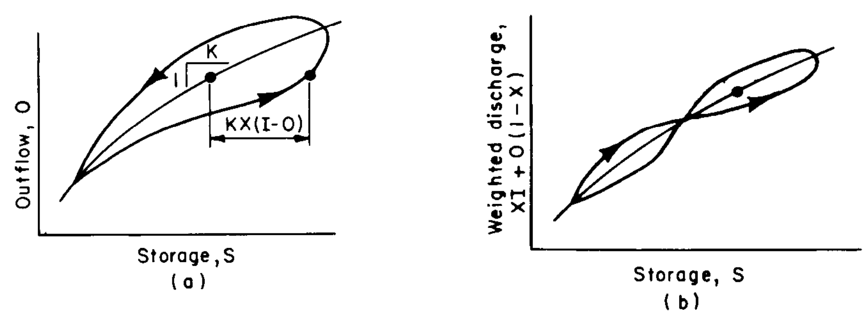


Figure 5-11.—Application of channel storage equation. 103-D-1926.

center of mass of the flood wave, of a selected discharge on the recession curve of the hydrographs, of the midordinate discharge of the rising leg of the hydrographs, or of some other characteristic point on the hydrographs.

(2) The celerity of the elementary discharge wave, V_w, can be approximated from the discharge rating curve of a station whose cross section is representative of the reach by using the following equation:

$$V_w = \frac{dQ}{B\, dy} \tag{12}$$

where:

V_w = celerity of elementary discharge wave,

$\frac{dQ}{dy}$ = slope of rating curve, and

B = breadth of channel at water surface.

By evaluating dQ/dy by the Manning Equation, equation (6) in section 5.5, the ratios between V_w and the mean velocity V for various types of channels would be:

Type of Channel	V_w/V
Wide rectangular	1.67
Wide parabolic	1.44
Triangular	1.33

The value of the mean velocity V may be obtained from the discharge and cross-sectional area of the representative section. The value of K is then the ratio of reach length to wave celerity V_w. Equation (12) becomes less applicable as the wave height increases, and therefore as the selected Δt value increases. The ratios in the previous tabulation were derived for a condition of constant slope, and therefore do not apply to the reach of a river entering a reservoir where slopes pivot on the pool level.

(3) If the basic data consist of a discharge rating curve at the downstream end of the reach, numerous cross sections in the reach, and a roughness coefficient, the K value will be approximately equal to the slope of a curve that represents the volumes under computed steady-flow profiles versus corresponding outflows. Obviously, this curve and K can also be determined if the storage is computed from sounding surveys for several steady-flow profiles.

(4) The K value may also be determined from actual hydrographs using an inverse process of flood routing. This method is preferable

to the three preceding methods because it provides a simultaneous determination of X and K, and because K represents the change of total storage per unit of weighted discharge, as shown by equation (11). For this method, equations (9) and (10) are solved for K to obtain:

$$K = \frac{0.5\ \Delta t\ [(I_2 + I_1) - (O_2 + O_1)]}{X\ (I_2 - I_1) + (1 - X)\ (O_2 - O_1)} \tag{13}$$

Equation (13) is described in a publication [67] by the Engineer School in Fort Belvoir, Virginia, as follows:

> ". . . successive values of the numerator and denominator are accumulated for floods for which the inflow and outflow are known, with X as a parameter. The accumulated numerator values are plotted as abscissae and the accumulated denominator values as ordinates. The result is a series of curves for various values of X. The one approaching more nearly to a straight line for the entire flood satisfies most closely the equation, and therefore determines the proper X for the reach. The K value is the reciprocal of the slope of the curve. Because of the limits of accuracy of the runoff data usually available, it is preferable to compute K and X for several floods and to adopt for routing an average of these values."

The latter method is illustrated in table 5-8 and on figure 5-12, which are based on material shown in reference [62]. In table 5-8, column (2) represents the inflow into the reach. Runoff values for the main stream and one tributary were obtained from USGS records, and the ungauged tributary runoff was approximated from distribution graphs and rainfall records. The total inflow was adjusted to equal the outflow volume. The numerator and denominator of equation (13) were evaluated for each period using four assumed values of X. On figure 5-12, the accumulated numerator (storage) values in column (9) of table 5-8 are plotted against the corresponding accumulated denominator (weighted discharge) values shown in columns (11), (13), (15), and (17) of table 5-8. The best fit for the curves shown on figure 5-17 is assumed to be that for which there is the least variation of the valley storage curve from a single line passing through it. The best fit appears to be for $X = 0.2$ and $K = 1.00$, the mean line being taken as straight throughout the range of discharges. To conform to a criteria that $K = \Delta t = 0.5$ day, the reach was subdivided into two equal reaches, and the value of K for each reach would be 0.5 day, assuming a constant wave celerity for the two reaches. However, X would not necessarily retain the value of 0.2. The ratio of reach length to K is the rate of flood wave movement, which can be applied to reaches of shorter or longer length unless there is a marked difference in storage characteristics between the desired reach and the reach for which the K value was derived.

The determination of K by any of the four preceding methods will result in a K value that usually varies with stage. A plotted curve of K values

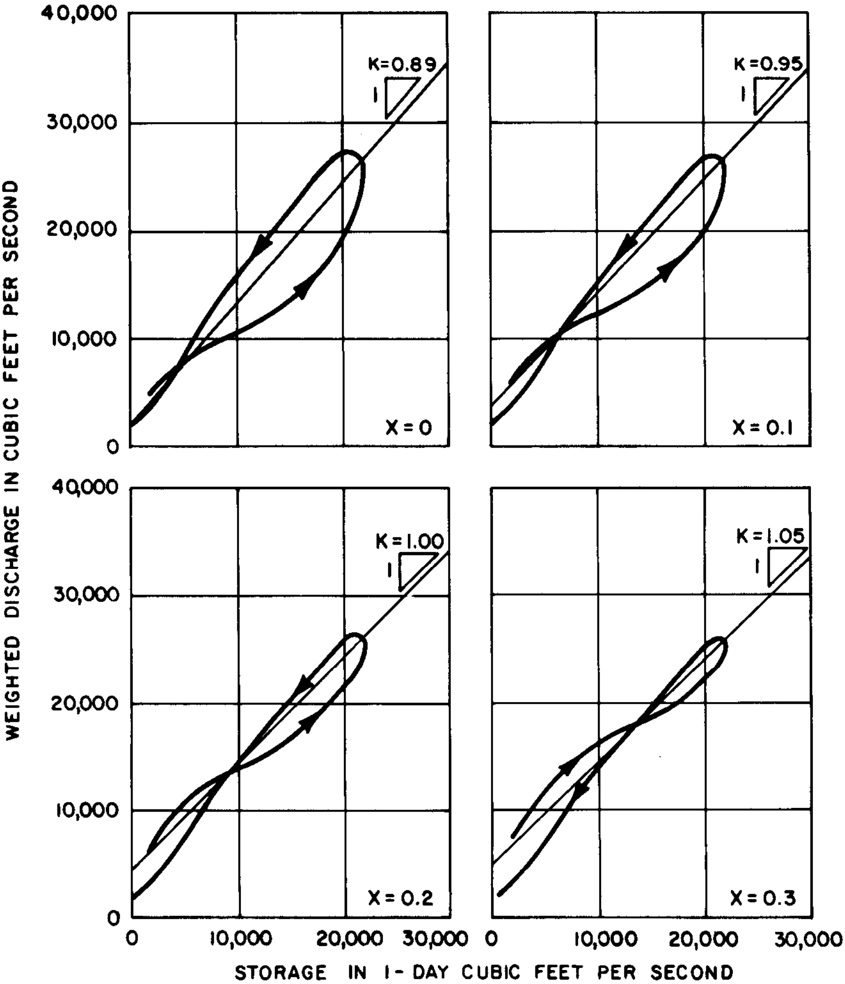


Figure 5-12.—Typical valley storage curves. 103-D-1927.

versus outflow may be prepared and, if a specific routing for a single value of K is to be selected, this K value may be taken as the value corresponding to the average of the initial flow and anticipated peak outflow.

The loop in the storage curves shown on figures 5-11(a) and 5-12 for $X = 0$ may result in part from the fact that discharge generally is not a single-valued function of stage but depends on the rate of rise or fall of the stage hydrograph. If discharges derived from average rating curves are corrected for this rise or fall, the corrected discharges would usually result in loops of lesser size. If trial values of X are applied to the corrected

Table 5-8.—Determination of coefficients K and X for the Muskingum Routing Method. Tuscarawas River, Muskingum Basin, Ohio Reach from Dover to Newcomerstown, February 26 to March 4, 1929.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	Values of D and ΣD for Assumed Values of X							
Date Δt= 0.5 day	Inflow[1], ft³/s	Outflow[2], ft³/s	$I_2 + I_1$, ft³/s	$O_2 + O_1$, ft³/s	$I_2 - I_1$, ft³/s	$O_2 - O_1$, ft³/s	[3]N	ΣN	$X = 0$		$X = 0.1$		$X = 0.2$		$X = 0.3$	
									[4]D (10)	ΣD (11)	D (12)	ΣD (13)	D (14)	ΣD (15)	D (16)	ΣD (17)
2-26-29 a.m.	2,200	2,000	16,700	9,000	12,300	5,000	1,900		5,000		5,700		6,500		7,200	
p.m.	14,500	7,000	42,900	18,700	13,900	4,700	6,100	1,900	4,700	5,000	5,600	5,700	6,500	6,500	7,500	7,200
2-27-29 a.m.	28,400	11,700	60,200	28,200	3,400	4,800	8,000	8,000	4,800	9,700	4,600	11,300	4,500	13,000	4,300	14,700
p.m.	31,800	16,500	61,500	40,500	−2,100	7,500	5,200	16,000	7,500	14,500	6,700	15,900	5,600	17,500	4,600	19,000
2-28-29 a.m.	29,700	24,000	55,000	53,100	−4,400	5,100	500	21,200	5,100	22,000	4,100	22,600	3,200	23,100	2,300	23,600
p.m.	25,300	29,100	45,700	57,500	−4,900	−700	−2,900	21,700	−700	27,100	−1,100	26,700	−1,500	26,300	−2,000	25,900
3-01-29 a.m.	20,400	28,400	36,700	52,200	−4,100	−4,600	−3,900	18,800	−4,600	26,400	−4,600	25,600	−4,500	24,800	−4,400	23,900
p.m.	16,300	23,800	28,900	43,200	−3,700	−4,400	−3,600	14,900	−4,400	21,800	−4,300	21,000	−4,300	20,300	−4,200	19,500
3-02-29 a.m.	12,600	19,400	21,900	34,700	−3,300	−4,100	−3,200	11,300	−4,100	17,400	−4,000	16,700	−3,900	16,000	−3,900	15,300
p.m.	9,300	15,300	16,000	26,500	−2,600	−4,100	−2,600	8,100	−4,100	13,300	−4,000	12,700	−3,800	12,100	−3,600	11,400
3-03-29 a.m.	6,700	11,200	11,700	19,400	−1,700	−3,000	−1,900	5,500	−3,000	9,200	−2,800	8,700	−2,800	8,300	−2,600	7,800
p.m.	5,000	8,200	9,100	14,600	−900	−1,800	−1,400	3,600	−1,800	6,200	−1,700	5,900	−1,600	5,500	−1,600	5,200
3-04-29 a.m.	4,100	6,400	7,700	11,600	−500	−1,200	−1,000	2,200	−1,200	4,400	−1,200	4,200	−1,100	3,900	−900	3,600
p.m.	3,600	5,200	6,000	9,800	−1,200	−600	−1,000	1,200	−600	3,200	−600	3,000	−700	2,800	−800	2,700
3-05-29 a.m.	2,400	4,600	—	—	—	—	—	200	—	2,600	—	2,400	—	2,100	—	1,900

[1]Inflow to reach was adjusted to equal volume of outflow

[2]Outflow is the hydrograph at Newcomerstown

[3]Numerator, N, is $\Delta t/2$, column (4) − column (5)

[4]Denominator, D, is column (7) + X [column (6) − column (7)]

Note: From plottings of column (9) versus columns (11), (13), (15), and (17), the plot giving the best fit is considered to define K and X.

$$K = \frac{\text{Numberator, } N}{\text{Denominator, } D} = \frac{0.5\Delta t\,[(I_2 + I_1) - (O_2 + O_1)]}{X(I_2 - I_1) + (1-X)(O_2 - O_1)}$$

discharges according to the fourth method for determining K, the loops for the best fit condition of K and X would more nearly approach a single curve.

(b) Determination of* X *Constant.—The effect of the variation in X on the shape of a routed flood hydrograph is shown on figure 5-13. When Δt equals K, an X value of 0.5 results in a hydrograph translated through the reach without change in shape. An X value of zero produces reservoir-type storage routing.

The symbol X has been considered to represent a dimensionless constant that is an index of the wedge storage in a routing reach. If the drawing on figure 5-14 represents a unit width of a wide rectangular channel, the wedge storage and prism storage for this unit width can be expressed as shown on the figure. By substitution, it can be shown that:

$$X = \frac{O\Delta y}{2y_o\,(I-O)} \tag{14}$$

the variables of which are depicted graphically on figure 5-14.

By differentiating the Manning discharge equation, if the slope is assumed to be constant,

$$\frac{I-O}{\Delta y} = \frac{5}{3}\left(\frac{O}{y_o}\right) \tag{15}$$

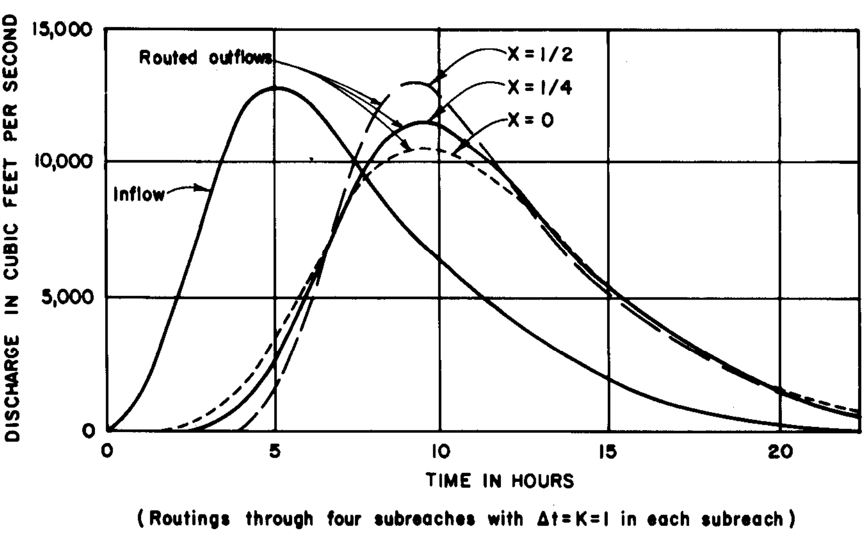


Figure 5-13.—Effects of a varying X constant on routed hydrographs. 103-D-1928.

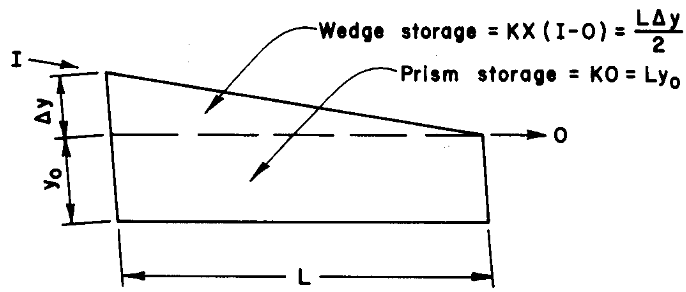


Figure 5-14.—Illustration of wedge and prism storage. 103-D-1929.

For a wide rectangular channel where changes in discharge are small and there is no variation in slope for such a change in discharge, X = 0.3 in equation (14). Under similar conditions, the X value for a triangular channel section increases uniformly from 0.375 at $\Delta y/y_o$ = 0 to 0.438 at $\Delta y/y_o$ = 0.5. Similiar evaluations for X can be made for other cross-sectional shapes. The significant point in these evaluations is that, within the limits of the noted assumptions, X depends primarily on the shape of the cross section and is relatively independent of river slope, roughness coefficient, and length of routing reach.

On figures 5-15 and 5-16, the hydrographs indicate that, when using the coefficient method of section 5.7, the value of X is not independent of reach length, as might be inferred from an analysis such as the one described above. On both of these figures, the hydrographs are shown for a reach that has been divided into subreaches for the purpose of routing. On figure 5-15, an X value of 1/4 applies to all routings. The hydrographs at the downstream end of the reach are shown to be dependent on the number and therefore the length of the subreaches. On figure 5-16, the effect of subreach length has been counterbalanced by varying the value of X. On figure 5-16, two hydrographs are represented within the width of line of the outflow hydrograph. The first hydrograph, for a routing through four subreaches with X = 1/4 and $\Delta t = K = 1$, is shown to be practically equivalent to a second hydrograph through eight subreaches with X = 0 and $\Delta t = K = 1/2$.

The previous paragraphs have discussed how to evaluate X from actual hydrographs. A second method consists of making trial routings with different values of X until one is found that satisfactorily reproduces the outflow hydrograph. This method may be necessary if the flood wave travel time in a reach between gauging stations exceeds $\Delta t/2X$. In this case, it is convenient to make the routings in n subreaches of travel time $K = \Delta t$ such that nK equals the travel time of the recorded data. This method is satisfactory on streams of relatively uniform cross section and slope, and having constant or small tributary inflows. A good example

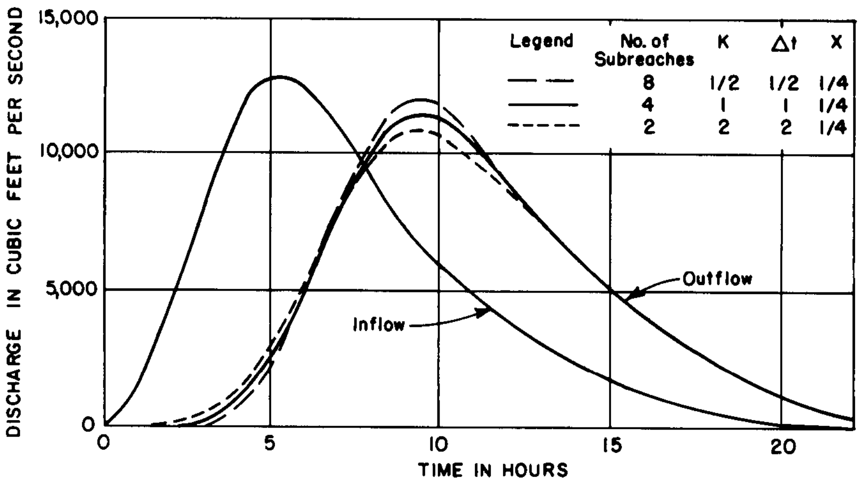


Figure 5-15.—Effects of a varying K constant and number of subreaches on routed hydrograph. 103-D-1930.

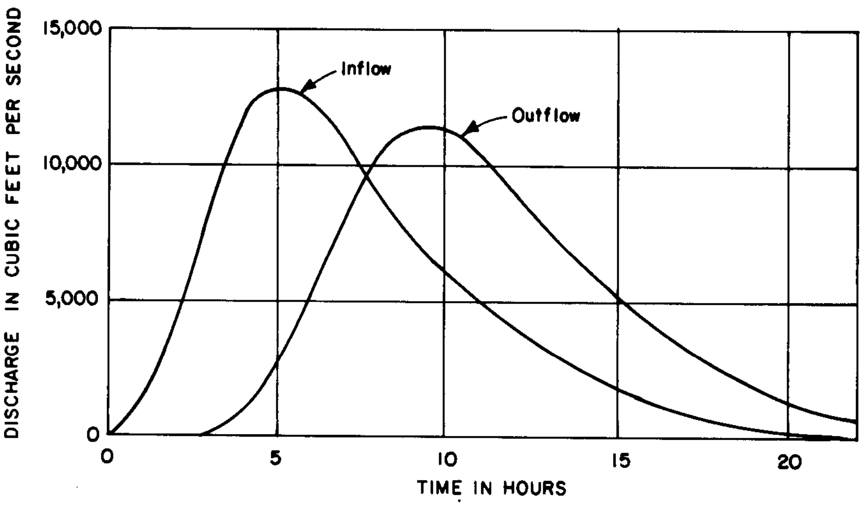


Figure 5-16.—Effect of subreach length counterblanced by a varying X constant. 103-D-1931.

of these conditions would be the Columbia River from the Grand Coulee gauge to the Trinidad gauge. Table 5-9 shows a trial routing through this reach for a minor rise in September 1945. The time for flood wave travel through the reach for this rise was 24 hours, and the routing was based on six subreaches. For each subreach, the values of Δt and K were both 4 hours. An X value of 0.3 resulted in a closer reproduction of the

Table 5-9.—Routing of Columbia River discharges[1] using routing constants derived by Muskingum Method.

September 1945		Discharge at Grand Coulee, ft³/s	Product of Inflow and Routing Constants[2]									Routed overflow, ft³/s	Local Inflow, ft³/s	Total Discharge at Trinidad, ft³/s	Actual Discharge at Trinidad, ft³/s
Day	Hour		[3]c_{n-2} =0.006	c_{n-3} =0.033	c_{n-4} =0.113	c_{n-5} =0.231	c_{n-6} =0.275	c_{n-7} =0.189	c_{n-8} =0.094	c_{n-9} =0.038	c_{RES} =0.020				
16	9	59													
	13	65													
	17	62	0*												
	21	63	0	2											
17	1	56	0	2	7										
	5	57	0	2	7	14									
	9	66	0	2	7	15	16								
	13	67	0	2	7	14	18	11							
	17	63	0	2	6	15	17	12	6						
	21	71	0	2	6	13	17	12	6	2					
18	1	93	0	2	7	13	15	12	6	2	1	58	6	64	66
	5	86	0	2	8	15	16	11	6	2	1	61	6	67	65
	9	94	1	2	7	15	18	11	5	2	1	62	6	68	67
	13	82	1	3	8	15	18	12	5	2	1	65	6	71	69
	17	77	1	3	11	16	17	13	6	2	1	70	6	76	72
	21	74	0	3	10	21	19	12	6	3	1	75	6	81	82
19	1	58	0	3	11	20	26	13	6	3	1	83	6	89	90
	5	54	0	3	9	22	24	18	7	2	1	86	6	92	92
	9	62	0	2	9	19	26	16	9	3	1	85	6	91	91
	13	59	0	2	8	18	23	18	8	4	1	82	6	88	88
	17	56	0	2	7	17	21	16	9	3	2	77	6	83	84
	21	58	0	2	6	13	20	15	8	4	2	70	6	76	79
20	1	56	0	2	7	12	16	14	7	3	2	63	6	69	73
	5	53	0	2	7	14	15	11	7	3	2	61	6	67	68
	9	61	0	2	6	14	17	10	5	3	2	59	6	65	66
	13	53	0	2	7	13	16	12	5	2	1	58	6	64	66
	17	53	0	2	6	13	15	11	6	2	1	56	6	62	63
	21	57	0	2	6	13	16	11	6	2	1	57	6	63	61

[1]Discharges are shown in 1,000 cubic feet per second.
[2]Routing constants, taken from table 5-10 for $X = 0.3$ and six subreaches.
[3]Values of c_n and c_{n-1} have so little effect on values on routed outflow that they were omitted from this table.
*The product of $Q = 59$ and $c_{n-2} = 0.006$ is offset by two lines, and the product in the next column is offset by three lines, etc. The summation of products horizontally yields the routed outflow.
Note: $X = 0.3$ and $\Delta t = K = 4$ hours for each of six subreaches.

measured hydrograph at Trinidad than X values of 0.2 and 0.4. The routing was made using the routing constants shown in table 5-10 because it was more convenient than routing through six subreaches by the more commonly used coefficient method.

It is always desirable to evaluate X from several sets of data. If no data are available, the selection of X must be based on judgment and derived from determinations on other streams; and allowances made for effects of reach length, flood wave travel time, and time interval. Also, it is desirable to retain the value of the time interval Δt that was used to determine X for any other applications of flood routing in the reach. If Δt is changed, the value of X must usually be changed.

Although X is usually taken as a constant for flood routing, it is evident from figures 5-11 and 5-12 that X may vary during a flood for a given reach. Also, the relationship between X and storage or outflow may vary from one flood to another. If the variable nature of X must be determined, it generally may be done by establishing, by trial and error, a reasonable curve of X versus outflow that will reduce the loops of the storage discharge curves to approximately a single line. It is occasionally necessary to derive X from data based on the amount of storage between steady and unsteady flow profiles. Flowline profiles in the reach may be determined by computations or field surveys initially for a constant flow and then for discharges increasing directly with the distance from one end of the reach. The storage computed from cross sections under the steady flow profile is the prism storage KO. The difference in the storages under the two profiles having the same discharge and stage at the downstream end of the reach is the wedge storage $KX(I - O)$. The complete determination for a reach will result in a family of curves relating O, I, and X. Generally, X varies more with O than with I, in which case X, like K, may be represented by a single curve showing X versus O.

The discussed method is sometimes the only practicable approach for developing X and K values for routing flow in a primary stream affected by tributary inflows or by other independent variables. For tributary inflow, curves of X and K versus O may be developed initially for the primary stream for several values of tributary inflow, and then for the tributary for several values of main stream discharges.

(c) Selection of Reach Lengths.—The ends of routing reaches should be selected with reference to the storage characteristics of the river valley. A section should be located at any channel control that creates relatively large storage upstream. For the extreme case, the storage-outflow relationship for such a reach is the reservoir type, for which $X = 0$, and the reach in which the reservoir-type storage is located should not be subdivided. A section should also be located at the upstream end of pool-type storage reaches. If a long stretch of river intervenes between pool-type reaches, the stretch should be subdivided at convenient sections,

Table 5-10.—Routing constants by Muskingum Method for reaches with different flood-wave travel times.

Routing Constant Symbol	Value of Routing Constant					
	$x = 0$	$x = 0.1$	$x = 0.2$	$x = 0.3$	$x = 0.4$	$x = 0.5$
	Reach with total flood-wave travel time equal to Δt					
Residual	0.010	0.004	0.001	0.001	0.000	0
c_{n-4}	.016	.012	.007	.003	.001	0
c_{n-3}	.049	.042	.032	.019	.007	0
c_{n-2}	.148	.146	.137	.116	.075	0
c_{n-1}	.444	.510	.592	.694	.826	1.000
c_n	.333	.286	.231	.167	.091	0
	Reach with total flood-wave travel time equal to $2\Delta t$					
Residual	.022	.012	.008	.003	.001	0
c_{n-5}	.033	.026	.018	.009	.002	0
c_{n-4}	.077	.071	.059	.041	.017	0
c_{n-3}	.165	.173	.176	.167	.125	0
c_{n-2}	.296	.344	.413	.521	.697	1.000
c_{n-1}	.296	.292	.273	.231	.150	0
c_n	.111	.082	.053	.028	.008	0
	Reach with total flood-wave travel time equal to $3\Delta t$					
Residual	.038	.025	.015	.005	.000	0
c_{n-6}	.045	.039	.029	.016	.004	0
c_{n-5}	.030	.087	.078	.060	.029	0
c_{n-4}	.159	.171	.183	.188	.159	0
c_{n-3}	.236	.271	.324	.417	.599	1.000
c_{n-2}	.247	.259	.264	.251	.188	0
c_{n-1}	.148	.125	.095	.058	.020	0
c_n	.037	.023	.012	.005	.001	0

Table 5-10.—Routing constants by Muskingum Method for reaches with different flood-wave travel times.—Continued

Routing Constant Symbol	Value of Routing Constant					
	$x = 0$	$x = 0.1$	$x = 0.2$	$x = 0.3$	$x = 0.4$	$x = 0.5$
	Reach with total flood-wave travel time equal to $4\Delta t$					
Residual	0.065	0.042	0.023	0.009	0.003	0
c_{n-7}	.054	.048	.039	.024	.007	0
c_{n-6}	.095	.095	.090	.075	.040	0
c_{n-5}	.150	.164	.180	.194	.180	0
c_{n-4}	.202	.230	.273	.351	.523	1.000
c_{n-3}	.212	.229	.244	.260	.211	0
c_{n-2}	.154	.141	.119	.083	.034	0
c_{n-1}	.066	.048	.029	.013	.002	0
c_n	.012	.007	.003	.001	.000	0
	Reach with total flood-wave travel time equal to $5\Delta t$					
Residual	.071	.052	.032	.013	.002	0
c_{n-8}	.060	.055	.047	.032	.011	0
c_{n-7}	.097	.099	.098	.086	.051	0
c_{n-6}	.142	.156	.173	.193	.193	0
c_{n-5}	.180	.204	.240	.307	.465	1.00
c_{n-4}	.188	.205	.225	.242	.225	0
c_{n-3}	.149	.144	.131	.101	.048	0
c_{n-2}	.082	.066	.045	.023	.005	0
c_{n-1}	.027	.017	.008	.003	.000	0
c_n	.004	.002	.001	.000	.000	0

Table 5-10.—Routing constants by Muskingum Method for reaches with different flood-wave travel times.—Continued

Routing Constant Symbol	Value of Routing Constant					
	$x = 0$	$x = 0.1$	$x = 0.2$	$x = 0.3$	$x = 0.4$	$x = 0.5$
	Reach with total flood-wave travel time equal to $6\Delta t$					
Residual	0.085	0.063	0.043	0.020	0.003	0
c_{n-9}	.064	.061	.053	.038	.014	0
c_{n-8}	.097	.101	.102	.094	.061	0
c_{n-7}	.134	.148	.166	.189	.201	0
c_{n-6}	.164	.185	.217	.275	.419	1.000
c_{n-5}	.170	.187	.208	.231	.232	0
c_{n-4}	.143	.143	.136	.113	.060	0
c_{n-3}	.091	.077	.058	.033	.009	0
c_{n-2}	.040	.028	.015	.006	.001	0
c_{n-1}	.011	.006	.002	.001	.000	0
c_n	.001	.001	.000	.000	.000	0
	Reach with total flood-wave travel time equal to $7\Delta t$					
Residual	.099	.077	.051	.027	.006	0
c_{n-10}	.066	.064	.058	.044	.018	0
c_{n-9}	.096	.101	.104	.100	.070	0
c_{n-8}	.128	.141	.159	.184	.205	0
c_{n-7}	.152	.171	.200	.252	.382	1.000
c_{n-6}	.157	.173	.194	.220	.234	0
c_{n-5}	.136	.139	.137	.121	.071	0
c_{n-4}	.094	.084	.068	.042	.013	0
c_{n-3}	.050	.037	.023	.009	.001	0
c_{n-2}	.018	.011	.005	.001	.000	0
c_{n-1}	.004	.002	.001	.000	.000	0
c_n	.000	.000	.000	.000	.000	0

Note: $O_n = c_n I_n + c_{n-1} I_{n-1} + c_{n-2} I_{n-2} + c_{n-3} I_{n-3} + \ldots$

including gauging stations, in accordance with the criteria requiring K to approximately equal Δt. Water-surface profile charts are useful in locating channel controls and inflow sections for pool-type reaches. If a major tributary joins the stream, the downstream end of the routing reach should be at or near the confluence.

5.8 Foss Dam Example

The example study on Foss Dam presented in chapter 4 had four subbasin flood hydrographs that had been developed, but they still needed to be routed and combined to obtain the probable maximum inflow flood. Referring to the basin map shown on figure 4-15, the flood hydrograph for subbasin 4 is channel routed from control point D to control point C, where the routed hydrograph is combined with the hydrograph for subbasin 3. Then, the combined flood hydrograph is channel routed from control point C to control point B. The combined and routed flood hydrograph is then combined with the flood hydrograph for subbasin 2, yielding the total hydrograph at control point B. The total hydrograph at control point B is then routed to control point A (Foss Dam). The total PMF hydrograph at Foss Dam is the sum of the hydrograph routed from control point B and the flood hydrograph for subbasin 1. This process is shown in table 5-11.

The channel routing method used for this example is called the Tatum Routing Method, which requires that a travel time through each routing reach be determined. This time was determined using Manning's equation (6) to determine velocities of flow. In this case, a Manning's n value of 0.03 was used. It should be noted that this n value is for the main stem of the Washita River only, the K_n value of 0.069 used in the unit hydrograph derivation reflects the hydraulic efficiency of the entire drainage network including overland flow, and would reasonably have a higher value than the main stem itself. Using topographic data to determine the main stem channel's cross-sectional area, wetted perimeter, and channel routing length, the following information was developed for each reach:

Control Point to Control Point	*Travel Time, hours*	*No. of Tatum Routing Steps*
D to C	12.6	13
C to B	5.1	5
B to A	3.2	3

After computing the travel time, the number of routing steps is determined by dividing the travel time, in hours, by the time increment of the flood hydrograph, for this case, 1 hour. Using table 5-3 or equation (4), for the reach from control point D to C, the routing constants to be applied to the inflow hydrograph ordinates to determine the outflow hydrograph ordinates can be determined.

Table 5-11.—Total PMF hydrograph for Foss Dam.

Time, hours	Subbasin 4 Hydrograph Ordinates at Control Point D	Subbasin 4 Hydrograph Ordinates Routed to Control Point C	Subbasin 3 Hydrograph Ordinates at Control Point C	Combined Hydrograph at Control Point C Sum of Columns (3) and (4)	Combined Hydrograph at Control Point C Routed to Control Point B	Subbasin 2 Hydrograph Ordinates at Control Point B	Combined Hydrograph at Control Point B Sum of Columns (6) and (7)	Combined Hydrograph at Control Point B Routed to Control Point A	Subbasin 1 Hydrograph Ordinates at Control Point A (Foss Dam)	Total Basin Hydrograph Ordinates at Control Point A (Foss Dam Inflow)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
37	0	0	0	0	0	0	0	0	0	0
38	0	0	10	10	0	0	0	0	0	0
39	0	0	30	30	0	20	20	0	0	0
40	10	0	130	130	0	140	140	0	60	60
41	50	0	380	380	0	540	540	30	360	390
42	190	0	950	950	20	1,550	1,570	110	1,320	1,430
43	610	0	2,060	2,060	50	3,720	3,770	380	3,670	4,050
44	1,740	0	4,280	4,280	150	7,800	7,950	1,050	8,610	9,660
45	4,440	0	8,390	8,390	370	14,800	15,200	2,500	18,000	20,500
46	9,960	0	15,700	15,700	850	27,800	28,600	5,330	35,800	41,200
47	20,200	0	27,600	27,600	1,800	49,900	51,700	10,600	66,600	77,200
48	36,800	0	47,600	47,600	3,620	87,000	90,600	20,000	113,700	133,700
49	61,600	10	77,300	77,300	6,920	136,700	143,700	36,200	170,800	207,000
50	90,800	40	118,900	118,900	12,600	193,500	206,100	62,300	228,400	290,700
51	119,500	110	169,800	169,900	22,100	243,300	265,400	100,500	270,600	371,100
52	139,400	290	222,100	222,400	37,100	276,200	313,300	149,500	273,900	423,400
53	147,600	680	265,200	265,900	59,300	250,500	309,800	203,800	222,100	425,900
54	129,200	1,510	291,500	293,000	89,800	197,500	287,300	253,100	170,000	423,100
55	102,600	3,100	278,300	281,400	127,800	155,200	283,000	285,700	138,400	424,100
56	81,500	5,990	232,500	238,500	170,300	130,300	300,600	296,900	117,700	414,600
57	68,400	10,800	187,600	198,400	211,200	113,000	324,200	295,500	101,600	397,100
58	60,300	18,200	155,000	173,200	242,600	99,300	341,900	295,900	88,400	384,300
59	54,000	28,800	134,600	163,400	257,600	87,800	345,400	305,500	77,100	382,600
60	48,600	42,500	121,200	163,700	253,600	77,800	331,400	320,500	67,200	387,700

Table 5-11.—Total PMF hydrograph for Foss Dam.—Continued

Time, hours	Subbasin 4 Hydrograph Ordinates at Control Point D	Subbasin 4 Hydrograph Ordinates Routed to Control Point C	Subbasin 3 Hydrograph Ordinates at Control Point C	Combined Hydrograph at Control Point C Sum of Columns (3) and (4)	Combined Hydrograph at Control Point C Routed to Control Point B	Subbasin 2 Hydrograph Ordinates at Control Point B	Combined Hydrograph at Control Point B Sum of Columns (6) and (7)	Combined Hydrograph at Control Point B Routed to Control Point A	Subbasin 1 Hydrograph Ordinates at Control Point A (Foss Dam)	Total Basin Hydrograph Ordinates at Control Point A (Foss Dam Inflow)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
61	43,800	58,700	110,500	169,200	234,700	68,900	303,600	332,200	58,600	390,800
62	39,400	75,900	101,300	177,200	210,100	61,100	271,200	333,600	51,100	384,700
63	35,500	91,700	92,900	184,600	188,800	54,200	243,000	322,000	44,500	366,500
64	31,900	103,700	85,300	189,000	175,700	48,000	223,700	300,100	38,800	338,900
65	28,800	110,100	78,300	188,400	171,000	42,500	213,500	273,600	33,800	307,400
66	25,900	110,000	71,800	181,800	172,400	37,700	210,100	248,800	29,500	278,300
67	23,300	104,300	65,900	170,200	176,800	33,400	210,200	230,000	25,700	255,700
68	21,000	94,800	60,500	155,300	181,200	29,600	210,800	218,400	22,400	240,800
69	18,900	83,700	55,600	139,300	183,400	26,200	209,600	212,700	19,500	232,200
70	17,000	72,800	51,000	123,800	181,900	23,200	205,100	210,700	17,000	227,700
71	15,300	63,100	46,800	109,900	176,100	20,600	196,700	209,600	14,900	224,500
72	13,800	55,200	43,000	98,200	166,300	18,200	184,500	207,300	13,000	220,300
73	12,400	48,700	39,400	88,100	153,700	16,100	169,800	202,300	11,300	213,600
74	11,200	43,300	36,200	79,500	139,600	14,300	153,900	194,000	9,860	203,900
75	10,100	38,800	33,200	72,000	125,400	12,700	138,100	182,600	8,590	191,200
76	9,060	34,900	30,500	65,400	112,200	11,200	123,400	168,900	7,500	176,400
77	8,160	31,400	28,000	59,400	100,300	9,950	110,300	153,900	6,550	160,400
78	7,360	28,300	25,700	54,000	89,900	8,840	98,700	138,800	5,720	144,500
79	6,630	25,400	23,600	49,000	80,900	7,820	88,700	124,500	5,020	129,500
80	5,970	22,900	21,600	44,500	73,100	6,950	80,000	111,400	4,430	115,800
81	5,380	20,600	19,900	40,500	66,200	6,160	72,400	99,900	3,900	103,800
82	4,840	18,600	18,200	36,800	60,100	5,470	65,600	89,700	3,410	93,100
83	4,360	16,700	16,700	33,400	54,600	4,880	59,500	80,800	2,950	83,800
84	3,920	15,100	15,400	30,500	49,600	4,350	53,900	73,000	2,460	75,500

Table 5-11.—Total PMF hydrograph for Foss Dam.—Continued

Time, hours (1)	Subbasin 4 Hydrograph Ordinates at Control Point D (2)	Subbasin 4 Hydrograph Ordinates Routed to Control Point C (3)	Subbasin 3 Hydrograph Ordinates at Control Point C (4)	Combined Hydrograph at Control Point C Sum of Columns (3) and (4) (5)	Combined Hydrograph at Control Point C Routed to Control Point B (6)	Subbasin 2 Hydrograph Ordinates at Control Point B (7)	Combined Hydrograph at Control Point B Sum of Columns (6) and (7) (8)	Combined Hydrograph at Control Point B Routed to Control Point A (9)	Subbasin 1 Hydrograph Ordinates at Control Point A (Foss Dam) (10)	Total Basin Hydrograph Ordinates at Control Point A (Foss Dam Inflow) (11)
85	3,530	13,600	14,100	27,700	45,100	3,900	49,000	66,100	1,980	68,100
86	3,180	12,200	13,000	25,200	41,000	3,490	44,500	59,900	1,470	61,400
87	2,870	11,000	11,900	22,900	37,200	3,100	40,300	54,400	990	55,400
88	2,590	9,890	10,900	20,800	33,900	2,720	36,600	49,300	340	49,600
89	2,350	8,910	10,000	18,900	30,800	2,320	33,100	44,800	90	44,900
90	2,140	8,020	9,200	17,200	28,000	1,910	29,900	40,600	20	40,600
91	1,950	7,220	8,440	15,700	25,500	1,470	27,000	36,800	0	36,800
92	1,770	6,510	7,730	14,200	23,100	1,030	24,100	33,300	0	33,300
93	1,600	5,860	7,110	13,000	21,000	410	21,400	30,100	0	30,100
94	1,440	5,280	6,510	11,800	19,100	120	19,200	27,100	0	27,100
95	1,270	4,750	5,980	10,700	17,400	40	17,400	24,300	0	24,300
96	1,100	4,280	5,490	9,770	15,800	10	15,800	21,700	0	21,700
97	900	3,860	5,040	8,900	14,400	0	14,400	19,500	0	19,500
98	690	3,480	4,660	8,140	13,100	0	13,100	17,600	0	17,600
99	470	3,140	4,310	7,450	11,900	0	11,900	16,000	0	16,000
100	210	2,830	4,000	6,830	10,900	0	10,900	14,500	0	14,500
101	60	2,560	3,720	6,280	9,890	0	9,890	13,200	0	13,200
102	10	2,320	3,450	5,770	9,020	0	9,020	12,000	0	12,000
103	0	2,100	3,180	5,280	8,240	0	8,240	10,900	0	10,900
104	0	1,900	2,910	4,810	7,540	0	7,540	9,970	0	9,970
105	0	1,710	2,660	4,370	6,910	0	6,910	9,090	0	9,090
106	0	1,520	2,410	3,930	6,330	0	6,330	8,300	0	8,300
107	0	1,340	2,180	3,520	5,800	0	5,800	7,590	0	7,590
108	0	1,160	1,920	3,080	5,310	0	5,310	6,950	0	6,950

Table 5-11.—Total PMF hydrograph for Foss Dam.—Continued

Time, hours (1)	Subbasin 4 Hydrograph Ordinates at Control Point D (2)	Subbasin 4 Hydrograph Ordinates Routed to Control Point C (3)	Subbasin 3 Hydrograph Ordinates at Control Point C (4)	Combined Hydrograph at Control Point C Sum of Columns (3) and (4) (5)	Combined Hydrograph at Control Point C Routed to Control Point B (6)	Subbasin 2 Hydrograph Ordinates at Control Point B (7)	Combined Hydrograph at Control Point B Sum of Columns (6) and (7) (8)	Combined Hydrograph at Control Point B Routed to Control Point A (9)	Subbasin 1 Hydrograph Ordinates at Control Point A (Foss Dam) (10)	Total Basin Hydrograph Ordinates at Control Point A (Foss Dam Inflow) (11)
109	0	970	1,620	2,590	4,840	0	4,840	6,370	0	6,370
110	0	780	1,260	2,040	4,380	0	4,380	5,830	0	5,830
111	0	600	890	1,490	3,940	0	3,940	5,330	0	5,330
112	0	430	280	710	3,490	0	3,490	4,850	0	4,850
113	0	290	80	370	3,030	0	3,030	4,390	0	4,390
114	0	180	30	210	2,530	0	2,530	3,940	0	3,940
115	0	100	10	110	2,000	0	2,000	3,480	0	3,480
116	0	50	0	50	1,460	0	1,460	3,010	0	3,010
117	0	20	0	20	970	0	970	2,510	0	2,510
118	0	10	0	10	590	0	590	2,000	0	2,000
119	0	0	0	0	330	0	330	1,500	0	1,500
120	0	0	0	0	170	0	170	1,040	0	1,040
121	0	0	0	0	90	0	90	670	0	670
122	0	0	0	0	40	0	40	400	0	400
123	0	0	0	0	20	0	20	230	0	230
124	0	0	0	0	10	0	10	120	0	120
125	0	0	0	0	0	0	0	60	0	60
126	0	0	0	0	0	0	0	30	0	30
127	0	0	0	0	0	0	0	10	0	10
128	0	0	0	0	0	0	0	0	0	0

* Time increments correspond to those tabulated in tables 4-23 through 4-26.

Chapter 6
ENVELOPE CURVES OF RECORDED FLOOD DISCHARGES

6.1 General Considerations

To provide the hydrologic engineer with information regarding flood potential in the area being studied, each flood hydrology study should include information relative to flood peaks and volumes that have been experienced in the hydrologic and meterologic homogeneous region. This information should be presented in the form of a curve that envelops the data points representing observed or recorded peak discharges or volumes for specified time durations versus the drainage area contributing to the observed flood runoff. In most studies, volume relationships will represent durations of 1, 3, 5, 10, and 15 days because the PMF hydrograph duration seldom exceeds 15 days. Exceptions are encountered in studies involving large drainage basins where the snowmelt flood component may extend from 1 to 2 months, as on the Colorado River at Glen Canyon Dam in Arizona. Figure 6-1 shows a typical example of the envelope curve relationship for peak discharges.

Envelope curves are of particular value to the hydrologic engineer in the development of PMF estimates because they provide information with which to judge the adequacy of those estimates. Considering the limited data base from which they are derived, relationships representing recorded flood events should never be construed as indicating an upper limit of the magnitude of future flood events. These relationships simply represent flood discharges that have been recorded or observed. As time progresses and more flood data are collected, each envelope curve will inevitably be altered in the upward direction. Eventually, the envelope curve will tend to approach the PMF as an upper limit. The PMF peak and volume values should always be higher than properly developed envelope curves; if not, both the data on which the curve is based and the PMF estimate should be carefully reviewed to determine if some hydrologic or meteorologic parameter has either been neglected or improperly evaluated and selected. If these parameters are judged to be adequate, the unique hydrologic or meteorologic feature that provides the apparently low estimate should always be identified and fully discussed in the flood study report.

When preparing envelope curves, the hydrologic engineer must ensure that the flood values used represent similar type flood events with respect to their meteorologic causes. There are four primary meteorologic causes that should be recognized and the data segregated accordingly: (1) thunderstorm type events where the resulting flood is caused by high intensity,

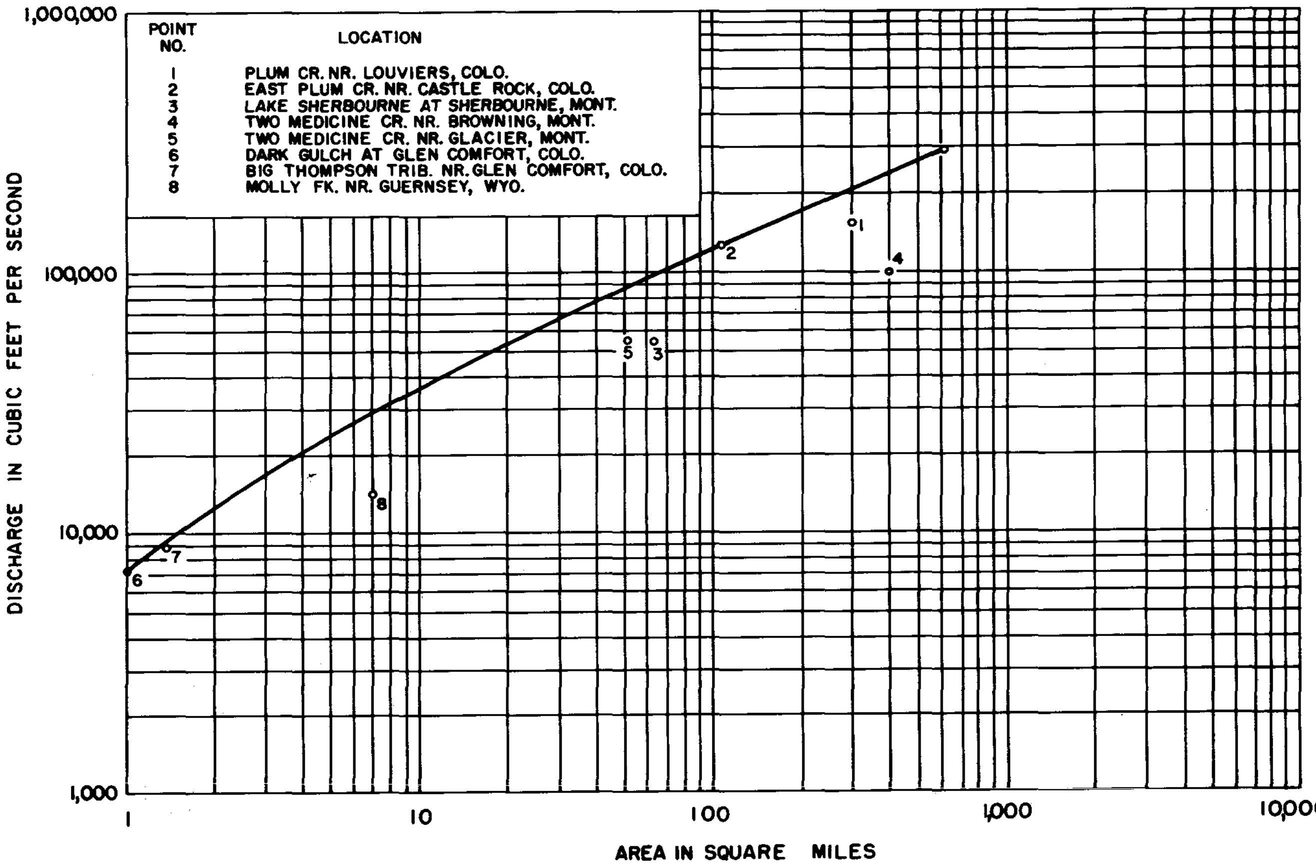


Figure 6-1.—Typical peak discharge envelope curve. 103-D-1932.

short duration, rainfall that produces high peak discharges and relatively low volumes; (2) general rain type events where the resulting flood is caused by moderate intensity, long duration, rainfall; (3) snowmelt floods resulting from the melting of an accumulated snow pack; and (4) floods resulting from a combination of rain falling on a melting snowpack. Each peak and volume envelope curve presented in the flood study report should provide information as to the causative factor involved.

Equally as important as meteorologic causative factors, the hydrologic engineer must assure that similar type drainage basins are represented from the hydrologic standpoint. For example, it is usually improper to include data representing flood runoff from steep mountainous drainage basins with those representing flood runoff from drainage basins in low relief plains regions. A severe storm will frequently cover only part of a large basin, but will still produce an extremely high flood. In these cases, the drainage area used in the development of the envelope curves is that of the storm area contributing to the flood runoff, not the entire drainage basin area above the location at which the runoff measurement is made.

6.2 Sources of Data

As previously discussed in chapter 2, the primary sources of basic data used to develop envelope curves are the water supply papers published by USGS. Of particular importance in the development of these curves are those papers in the general series titled "Magnitude and Frequency of Floods in the United States." The volumes comprising this series provide summary tabulations of peak discharges at the national USGS stream gauging network up to the year the particular volume was prepared for publication. For subsequent years, the records contained in annual water supply papers for stream gauges in the region under investigation should be closely examined to determine if the values contained in the aforementioned series have been exceeded and, if so, the more recent values should be included in the data set used to develop the peak discharge envelope curve. Data for developing volume envelope curves are contained in the annual water supply papers.

The computer program WATSTORE mentioned in section 2.2(a) of chapter 2 may also be used to obtain lists of peak discharge values and volumes for specified durations for a specific geographic region. These lists are obtained by inputing the latitude and longitude of the point of interest and the radius of a circle around that point within which all available records are desired. The output will then provide all the available data.

Many State governments, generally through either their water resource agency or highway department, have installed networks of crest stage gauges. These gauges, that provide information from which peak discharges can be determined as described in chapter 2, are installed to provide the basic flood data used in establishing design criteria for highway cross drainage structures such as culverts, boxes, and bridges. Records of peak discharges at these gauges are published at various intervals by the States involved. The hydrologic engineer should contact the appropriate agency with jurisdiction in the area of interest to determine if such data are available. If data are available, it should be obtained for use in developing peak discharge envelope curves. These data have generally been found to provide a valuable supplement to the systematic data acquired and published by the USGS. Other data sources would be the various reports prepared by the Bureau, COE, NWS, and some local governments that provide considerable information on observed flood events.

6.3 Procedure

The procedure for developing envelope curves is relatively simple and straight forward. Initially, a small scale map should be obtained for plotting the limits of geographical area reflecting similar hydrologic characteristics and meteorologic phenomena. This map should be included in the flood study report. The next step is to determine the location of all streamflow gauging stations, both recording and crest stage, within the limited geographic area. These stations should be plotted on the map and properly identified with the conventional USGS station number or its name; e.g., Arkansas River at Pueblo, Colorado. In addition, a tabulation of the data used to develop the envelope curves should be prepared for inclusion in the flood study report. The table should arrange the data in tabular form under the following headings:

Column 1: List the identifying number as provided on the map. This is useful to cross reference the location of data points on the map and to identify their position on the envelope curve.

Column 2: Stream or river name as identified in water supply papers or other source.

Column 3: Location of gauge on river or stream as listed in water supply papers or other source.

Column 4: Identify source of data; e.g., USGS Water Supply Paper No. 1537.

Column 5: Drainage area in square miles. Where only the contributing area is used, list only that area and provide a table footnote to that effect.

Column 6: Date of flood event.

Column 7: Peak discharge in cubic feet per second of time or volume over a specified period.

A separate table should be prepared for the peak discharge relationship, and for each duration for which volumes are examined. Peak discharges are readily obtained from various sources and may be used directly in the development of peak discharge envelope curves. Data for the volume envelope curves are compiled from the annual water supply papers. The 1-day values may represent either a 24-hour clock time amount or a calendar day amount. In recent years, data have been provided from which the 24-hour amounts can be determined, while the older papers provide only daily amounts. The 1-, 3-, 5-, 10-, and 15-day volumes in all cases are to represent volumes associated with the peak discharges used for the peak discharge envelope curves.

The data listed in columns 4 and 7 are then plotted on log-log paper with sufficient cycles to cover the range in discharges, and the drainage or contributing area sizes represented by the data. In all cases, the drainage or contributing areas are to be plotted on the abscissa and the peak discharges or flood volumes on the ordinate. A smooth preliminary curve is then drawn that envelops the plotted data points on the high side, as illustrated on figure 6-1. At this point, it should be apparent that only a few of the total data points control the position of the preliminary envelope curve. The data for each point that controls the preliminary curve should then be rechecked to ensure the data represent flood runoff from basins of comparable topography, soils, vegetation, and meteorological characteristics to the basin being studied. If inconsistencies are found in this final check, the data point should be eliminated. After verifying that all control points are suitable, the final curve enveloping these points may be drawn.

Chapter 7
STATISTICS AND PROBABILITIES

7.1 Introduction

The objective of frequency analyses in flood hydrology is primarily to estimate the frequencies or probabilities of future flood events. The desired relationship is intended to provide an estimated flow corresponding to a given probability of occurrence. An inverse but equivalent relationship is used to indicate a probability associated with a given flood flow. The purpose of this chapter is to describe and illustrate the application of statistics to frequency analysis used in flood hydrology work. Primarily, peak flows are the subject of these analyses, therefore, the main orientation of this chapter is towards analyses using this type of data. The term "peak flows," generally means annual instantaneous peak flows, which are the largest flow rates that have occurred for each year of recorded events. This chapter also covers many of the situations that often require analysis of different types of flows or for different unit-time periods. The probabilities of concern are annual exceedance probabilities, which are the probabilities of a given flow level being exceeded once or more in any given year. These probabilities are best expressed as annual probabilities; however, the flow level is often equivalently identified as the n-year flood, where n is equal to one divided by the annual exceedance probability. For example, the 100-year flood is the flow level with a 0.01 annual exceedance probability.

The scope of this chapter is necessarily limited because volumes could be devoted to the many aspects involved in flood flow frequency analysis. Also, this chapter is not intended to be a treatise on statistics or statistical methods; there are many books written on these subjects for all levels of interest and background. A complete review of all flood frequency approaches is also not presented because to do so would require a book in itself. While this chapter does provide some excellent examples and guidance, it cannot be considered as a textbook on how to perform standard frequency analyses. For a good textbook-type approach, see Bulletin 17B [14][1]. This chapter is more of a guideline that provides reasoning and a "feel" for what is important. As such, theory will be presented only as it provides a basis for aiding in the application and interpretation of flood frequency results. A review of basic concepts and an introduction to further topics is presented, and examples are provided for probability relationships and some indication of their reliability. Hopefully, this chapter will provide some insight into the process involved, what is analyzed, and what is important to study. The material is not intended to be a substitute for theoretical background or experience.

[1]Numbers in brackets refer to entries in the Bibliography.

The magnitude and frequency of floods are very important to the proper design and operation of water resource projects. For this chapter, the orientation is directed towards Bureau practice in the design and planning of projects and in the analysis of existing dams under the Safety of Dams Program. While flood control benefit evaluation is a most obvious application, this topic will not be discussed because of the nature of Bureau projects and the Bureau's working relationship with COE, who has primary responsibility in determining these benefits. The Bureau's main uses for the derived relationships include diversions during construction, cross-drainage design, design of low hazard dam spillways, and antecedent events for design floods for high hazard dams, usually the PMF. The relationships may also aid in the design, selection, and siting of equipment and facilities such as pumping equipment, generation capability, diversion channel headworks, and sill elevations for spillways.

To further clarify the intentions of this chapter, it is oriented towards the practicing hydrologic engineer who is actively involved in the estimation of flood frequency relationships. The discussions are not oriented towards the researcher, and do not cover present or future trends in research. An attempt was made to provide some practical guidance as to what is important and what to expect for results. Hopefully, a knowledge of the general behavior of flood flow data will allow practitioners to benefit more from their past experiences.

Please note that statistics do not provide answers and do not prove theories; however, statistics can be used to verify that a hypothesis cannot be shown to be obviously unreasonable. Statistics can indicate obviously incomplete or unreasonable theories, but statictics cannot prove a theory to be correct. Statistics can also be used to aid in indicating areas or relationships that show promise; reason is required to defend and support any such relationships. To decide between competing relationships based only upon the statistical "fit" is not defendable. Small differences in statistical fit have frequently been used wrongly to reject one relationship over another. A serious mistake is often made when the rejected relationship can be defended with reason, while the accepted relationship cannot be. Statistics should not be used to contradict reason.

To complete the chapter, a section is devoted to the data that are being analyzed. Following that, the general philosophy of probability distributions and a basic background of their application to flood hydrology are presented. The log-Pearson Type III distribution, which is the standard for Federal practice, is then covered in a manner complimentary to that presented in Bulletin 17B [14]. A short section covers a vitally important topic, the limitations on frequency curve extrapolation. This is followed by three major subtopics: (1) mixed populations, (2) volume analysis, and (3) ungauged analysis. Having given a somewhat complete background, the next section presents some information on general relationships that are found in flood frequency analyses.

7.2 Hydrologic Data Samples

The data used in frequency analysis varies with regard to several important factors. The data set itself may be recorded in any of several time units, and may result from either direct or indirect measurement or estimation. The analysis to be performed also influences the choice of data to be used. In addition to the basic form of the data, the accuracy and homogeneity of the data deserve consideration. It should be emphasized that the recorded data set is only a limited sample of the total number of floods that have occurred.

(a) Type of Data.—The data used in frequency analysis may be of several forms. Usually, instantaneous peak flood values are used, which is the type of data that has been studied the most and the type for which most analysis techniques have been specifically developed. The data consist of a series of the instantaneous peak values that have one peak value for each year of record. The years are usually either water years, October of previous calendar year through the following September, or calendar years. In this chapter, the orientation is primarily towards instantaneous data and, unless otherwise stated, it may be assumed that the data involved are instantaneous.

Although instantaneous peak data are the main form analyzed, other forms of data are often used, either because of the unavailability of instantaneous data or because instantaneous data is not applicable to the study. Occasionally, daily valvues may be readily available, while instantaneous values are only available for a short time period. A relationship or correlation may be developed between the instantaneous peaks and the peak daily values. The longer record of daily values could then be used to fill in surrogate values for the instantaneous peaks. Alternatively, the frequency distribution of the daily values could be analyzed and the results used to estimate corresponding instantaneous values by using the derived relationship. The relationship could be a simple ratio between the instantaneous peaks and the peak daily values. In some cases, the ratio could be estimated using a similar nearby stream, or by using several streams in the region.

Frequently in a study, the instantaneous peak is not of critical importance. For example, a dam with considerable surcharge storage but a relatively small discharge capacity is often most critically stressed by a large volume flood rather than by a high-peaked flood. In this situation, a 5- or 10-day (or longer) flood volume may be of prime importance. Therefore, frequency estimates are often required for flood volumes over various time increments. Annual peak values are usually used with 3-, 5-, 10-, 15-, 30-, 90-, and 120-day volume values. In some cases, peak weekly, monthly, seasonal, and annual values could also be applicable. Obviously, annual peak 30-day volumes will not behave in the same statistical manner as instantaneous peak values. Not all of the statistical techniques will be equally applicable to all volume-type data and considerable extra care must be taken when using the volume data.

Most often, it is not clear what time period is most critical for a dam. In most cases, an entire hydrograph is needed. One approach that seems to overcome these problems with good results is the use of a "balanced flood hydrograph." The balanced flood hydrograph is a reasonably shaped hydrograph representing a given frequency that preserves an entire set of volume values for differing time periods. Depending on the exact problem existing, the instantaneous peak could also be preserved; however, this is often not appropriate and is not the usual case. If a balanced 100-year hydrograph using the results of a set of frequency analyses with 3-, 5-, and 10-day volumes was developed, the resulting hydrograph would preserve all three of the 100-year volume values. It is also possible, although rare, to start with a set of volume data that represent the peak events from each year. Each data element would be for the entire volume of the event, and the time basis would vary from event to event.

Peak 24-hour values could also be analyzed, although this would be a rather rare application. It should be noted that there is a subtle, but sometimes very important, difference between the peak 24-hour value and the peak daily value. The peak 24-hour value is the largest volume in any 24-hour period starting at any time during the day. The peak daily value is restricted to the largest volume in a 24-hour period that starts at midnight. Thus, the peak 24-hour value will always be larger than the peak calendar day value. The same situation is true when dealing with peak 30-day values as compared to peak monthly values.

(b) Data Source.—The source of the data is often the actual measurements taken at the site of interest. However, the data may also be estimated flows based on the transposition of flows from an upstream or downstream site on the same stream. This could also be estimated flows based on transposition from a nearby similar stream; however, this type of data is less desirable. At times, some of the data could be based on actual measurements while additional data is obtained from other sites. Regional relationships using data from several sites and based on the hydrologic attributes of the drainages could also be used to furnish synthetic data; however, this can be an involved process. Another source for the basic data could be through the modeling of the rainfall-runoff process. To be reasonably accurate, this type of data source requires a continuous modeling process with continuous rainfall data, either actual data or stochastically generated data.

Rainfall frequency data are occasionally used as input to an event modeling of the runoff process using the unit hydrograph approach discussed in chapter 4. This approach is usually less than satisfactory for the frequency estimation of floods because of the lack of a distinct correspondence between rainfall frequency and flood frequency. This lack of correspondence is due mainly to the effects of the initial soil conditions, which must be estimated. The rainfall-runoff modeling procedure will give a general indication of the variability and potential for floods, but the results should be carefully used only as a guide to floods having the same probability of exceedance as the rainfall being used. This procedure should be used only after careful study and after all other approaches have proven infeasible.

(c) Data Analysis.—The hydrologic data sample can vary with the type of analysis. Most analyses are performed using annual maximum values, which is the main orientation of this chapter. Other types of analyses that require other types of data include flow duration analysis, partial duration analysis, seasonal analysis, and analysis by flood type.

(1) *Flow duration analysis.*—This type of analysis is used to express the fraction or percentage of time that flows exceed various levels. It should be noted that this type of analysis is not intended for extraordinary peak flows (floods) but rather the more day-to-day flows. The results may be expressed as the proportion of the year that a given flow level is exceeded; for example, a flow of 200 cubic feet per second is exceeded 23 percent of the time. Results may also be expressed in terms of the number of exceedances per year; for example, a flow of 300 cubic feet per second is exceeded on a daily average basis five times per year. For the more rare type of flooding, the annual éxceedance probability using annual peak flow data and the exceedance frequency using the full daily peak flow series (flow duration analysis) will be very similar. Flow duration analysis is most often used when ordinary flow levels are of importance. Since this is rarely the concern in flood hydrology, this subject will not be discussed further.

(2) *Partial duration analysis.*—This type of analysis is similar to flow duration analysis in that more than one value is used per year. All flow values above some arbitrary level are included in the analysis. Usually, this level is set low enough to include at least one value in each year and will generally have an average of three to five values per year. Normally, only one value is included for each flooding event; i.e., a multiple-peaked event would only be counted once. This type of analysis is sometimes used where multiple flooding within a year is important. An example would be the case where losses are suffered with each and every occurrence. While this type of analysis is often of interest in quantifying flood control benefits, it is not usually used by the Bureau. The resulting frquencies for the more rare floods are naturally very similar to normal annual peak analysis procedures.

(3) *Seasonal analysis.*—Frequently, the flood potential must be evaluated on a seasonal basis. In some cases, the operation of a reservoir may make it more susceptible to floods at certain times of year. During construction, the site may only be vulnerable during a limited time span,

and it may be necessary to analyze the flood frequency on a seasonal basis. In this case, the data would consist of only data from the season in question. Some cautions need to be applied in this case because the resulting probabilities are no longer "annual" probabilities. Rather, the probabilities are the frequency or chance of exceedance in any given single season of the type of flood being studied. There is a possibility of understating the flood risk when using seasonal analysis. Protecting against the 1 in 100 seasonal flood for each season in the year would seem to provide the same protection as protecting against the 100-year flood; however, this is not the case. As an example, consider a situation where each year has two seasons of equal flooding potential with all other seasons having inconsequential flooding potential. The 1 in 100 seasonal floods may be equal for both seasons, assume 150 cubic feet per second. However, this flow is not the 100-year flood. In fact, it is probably much closer to the 50-year flood because, on the average, two floods should be expected, one for each season in a 100-year period. To properly evaluate the actual risks involved, it is necessary to evaluate the consequences for each season, quantify the seasonal distributions of those consequences, and combine those distributions into one single annual probability distribution. Naturally, this combining process is not just the simple addition of exceedance probabilities.

(4) *Analysis by flood type.*—In many instances, flooding on a given stream may be due to more than one type of flooding. For example, a stream may be characterized as having thunderstorm floods, floods from general storms, and snowmelt floods. These types of flooding may take place at different times of the year and each may exhibit different statistical behavior. This situation is referred to as a "mixed population condition," because one population consisting of all floods is really a mixture of the floods from distinctly different subpopulations. In this case, the raw flood data should be divided into a separate series for each type of flooding. To avoid understating the risk, care should be taken to evaluate the total risk rather than only the individual risk due to each flood type. A mixed population analysis is often required and does improve the flood frquency analysis. This subject is discussed further in section 7.7.

(d) Accuracy of Data.—For the application of flood frequency analysis, the basic data is usually assumed to be free of errors; that is, the flow values are assumed to be accurate estimates of the actual flows. However, there is the potential for errors, and one should be aware of the effect that these errors could have on the fitted distribution. Errors tend to be classified as being either random (no pattern) or systematic (having a pattern). Random errors, being sometimes high and sometimes low, tend to compensate and, although they increase the variability of the data, are generally considered to have only an insignificnt effect on the results. Conversely, systematic errors can be very significant. If the systematic error is a constant value, the effect is only significant for the smaller floods and usually of little concern; however, the systematic error may distort the frequency curve. If the error is a constant percentage error, it will be reflected in the same percentage change in all of the frequency curves, and will only distort the value of the log mean and not the slope or shape of the curve if the log-Pearson Type III distribution is used. Where possible, known changes in the basin response, such as the effects of an upstream reservoir, should be removed from the data series to make it homogeneous. A more subtle but potentially serious source of error is when the stage discharge relationship for a stream becomes less accurate with increasing flow. Since the prime interest is in the less frequent rare flood, the errors involved may be highly compounded due to the extrapolation of the frequency curve.

It should be noted that the assumption the data is error free does not mean that the data sample is truly representative of the population. Sampling variation is, however, more easily quantifiable using statistics.

7.3 Frequency Distributions

An understanding of the statistical concepts and philosophies is desirable, along with the knowledge of the traditional approaches that have been used with flood data. The intent of this section is to emphasize some of the more important ideas, not to provide a complete discussion.

(a) Concepts and Philosophy.—The basic premise behind all frequency analyses is that the process is in some way governed by laws of chance; that is, that the outcomes are not entirely predictable, but have some random behavior. In everyday life, there are risks involved in practically all activities, and these risks are implicitly accepted without carefully assessing the odds involved. Probability and statistics, however, are devoted to the study and calculation of the probabilities of incurring these risks. Usually, the interest is in the probabilities of failure. To be able to calculate risks, certain basic assumptions must be made. The first assumption is that the process be random in nature. This is obvious because if the process was not random in some way there would be nothing to study. Peak flow is a very random variable. Even if flows are considered to result from a deterministic (or semideterministic) process, the flows are determined as the result of other random inputs, such as precipitation and the initial conditions of the basin. As a result, flows are, by definition, also random. Annual instantaneous peak flows are assumed to be identically distributed from year to year. This is a necessary and reasonable assumption, although adjustments may be necessary in the case of changing basin conditions. This assumption implies that the basic process is also unchanged from year to year. Also, it is assumed that the values are statistically independent from year to year, and that these values are not dependent upon one another. The distribution of possible values is not altered by previous values. The assumptions of flows being independent, identically distributed random variables are basic to the frequency analyses discussed in this manual. These assumptions are easily justified for instantaneous peak flows but need careful examination when working with long duration volume values such as seasonal or annual runoff.

Probability is the branch of mathematics that deals with the calculation of risks based on known processes using the basic laws of chance. Statistics, however, involves processes that are not known, and these processes must be inferred from observations. Flood frequency analysis is, in fact, a statistical analysis. Once the properties have been inferred from the data and the process assumed to be known, the conclusions may be drawn. Statistics relies heavily upon probability, while probability does not require statistics.

Basic to all flood frequency analyses is the concept that there is a population of potential values that may become flood values. This population includes all of the values that could take place, and is the set of all possible results. For flood flows, this population is made up of continuous values, generally considered to be bounded on the low end with zero flow and unbounded at the upper end. These values follow a frequency distribution. Both past and future events are assumed to come from this distribution and that the process (distribution) is unchanging with time (stationary). The order of the events is random, therefore the events are independent.

The record of flood flows constitutes the data sample, which is a sample of the flood flow population. The sample is frequently called the "sample population," while the total flood flow population is called the "target population." The sample is made up of the actual flows that have been experienced and recorded in some manner. While drainage basin changes such as urbanization, agricultural use, or construction of water control facilities may affect the homogeneity of the record, this is usually not the case and the sample is assumed to be homogeneous. More importantly, the sample is assumed to be a representative sample of the population. From this data sample, the behavior of the entire population is inferred. The choice of the basic form of distrubtion of flood flows is based on the general behavior of numerous data samples from many streams and rivers. In a strict sense, the form of the distribution is also an assumption. Only the parameters of the assumed distribution are inferred from the sample data. The reliability with which the population can be inferred from the sample data is a function of how representative the data sample is. The size of the sample; i.e., number of years of record, is indicative of the reliance that can be placed on the sample. The larger the sample, the less the chance will be that the sample is unrepresentative. However, a larger sample is not automatically more representative than a smaller sample, but is more likely to be so.

From the previous discussions, it has been shown that the flood frequency approach relies on several fundamental assumptions:

- Nature follows laws of chance

- Process does not change with time
- Sample data are representative
- Population characteristics can be inferred from sample
- Sample data are actually results that are independent, identically distributed random variables
- A general form or framework for the probability distribution is involved

This form or framework could be the commonly known "normal distribution" or, more typically for floods, the "lognormal distribution" or "log-Pearson Type III distribution." These assumptions, while not always acknowledged, form the basis for the actual flood frequency analysis.

Generally, the analysis consists only of inferring the characteristics of the population based on the sample data. By inferring the characteristics of the population, what is generally meant is the estimation of the parameters of the probability distribution. A final step in the analysis, which is of considerable importance and should not be neglected, is the assessment of the uncertainties. This usually takes the form of an evaluation of the confidence in the results.

(b) Forms of Presentation.—The results of a frequency analysis are in the form of a relationship between the values and the probabilities associated with those values. Results may be in the form of charts, tables, or more commonly as curves. The results are usually in terms of the probability of an event taking place (discrete probability), probability of an event exceeding a certain value (exceedance probability), or the probability of an event not exceeding a certain value. For floods, the most common form of presentation is the exceedance probability.

The terms frequency and probability are not the same because frequency has to do with the counting of occurrences of some event within some stated time period, and probability is a statement concerning the chance of an event taking place during some specified period. Note that a frequency can exceed one, while a probability must be between zero (impossible to occur) and one (definitely to occur). As events become rare, frequency and probability are nearly the same, although it can be shown that the frequency will always exceed the probability no matter how rare the event. In flood hydrology, frequency is generally used synonymously with probability with the understanding that the meaning intended is, almost without exception, that associated with probability. Terms such as "flood frequency" and "frequency distribution" are used, but the actual intent is almost always "flood probability" and " probability distribution."

There are two main forms of curves used to present flood frequency distributions: (1) probability density function and (2) cumulative distribution function. The probability density function curve has the familiar "bell" or "hat" shaped look. The ordinate scale has units of probability density, probability per unit chnage in the variable, while the abscissa is in terms of the variable; the axes are usually Cartesian. This form of presentation is shown on figure 7-1. Integration of the area under this curve between two values of the variable yields the probability of having an outcome with a value between these two values. Naturally, the total area under the curve is equal to one. For many variables in our day to day environment, the distribution is nearly normal and symmetrical. For floods, the distribution is skewed to the right. The logarithms of the flood flow values tend to be more nearly symmetrical, and the log of the values are closely approximated to the normal distribution. Examples of skewed probability density function curves are shown on figure 7-2. The probability density function is difficult to construct, and while it conveys a good deal of knowledge about the more common events, it does not give much information about the tails of the distribution that are indicative of rare events. Since it is the tails of the distribution that are of

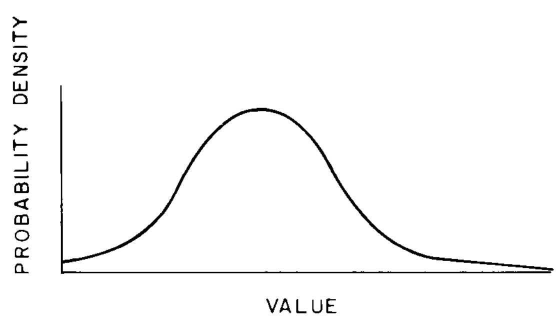


Figure 7-1.—Probability density function curve. 103-D-1933.

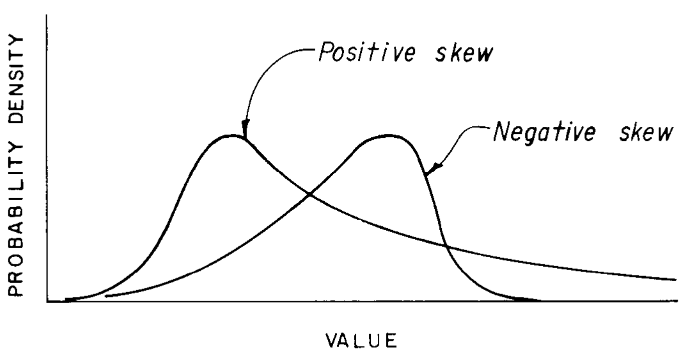


Figure 7-2.—Examples of skewed probability density function curves. 103-D-1934.

most interest in flood hydrology, the probability density function is not often used.

The cumulative distribution function is one that relates the exceedance probability to the value of the variable. In fact, this function is the integration of the area under the probability density function from the value in question to the largest value possible. The exceedance probability form is the most commonly used form for presentation of flood frequency distributions. In most statistics texts, however, the cumulative distribution function is defined in terms of nonexceedance; that is, the function gives the probability of an outcome less than or equal to the given value. It is easy to keep the two forms separate because each form is simply one minus the other. A typical cumulative distribution function is shown on figure 7-3. Usually, the abscissa represents the exceedance probability and the ordinate represents the variable value. However, the two axes are frequently switched. This form of presentation, which is fairly easy to construct, does not convey the meaning concerning the more common events as well as the probability density function, but it does better for the more extreme events. This form of presentation is ideal for flood hydrology because the concern is with the more rare events. As shown on figure 7-3, values for the cumulative distribution function range from zero to one. The graph paper used to present this curve is often constructed with a highly distorted axis to arrive at a curve that is as near to a straight line as possible. Usually, the abscissa is transformed from Cartesian to a normal probability scale that is constructed such that a normal probability distribution would plot as a straight line. This is accomplished by creating a Cartesian scale in terms of standard deviations of the normal distribution and then labeling the scale in terms of the corresponding exceedance probabilities. In addition to the distortion in the abscissa, the ordinate axis is also often "distorted" by using a logarithmic scale in this direction. When both scales are distorted, lognormal paper is created and a lognormal distribution would plot as

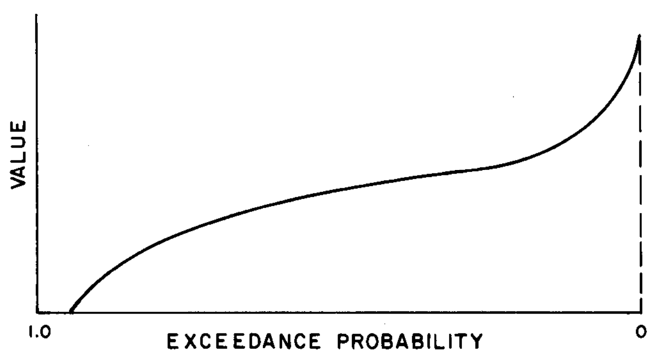


Figure 7-3.—A typical cumulative distribution function curve. 103-D-1935.

a straight line. Lognormal paper is very useful in that it facilitates the drawing of frequency curves and, for most flood data, the curves are more nearly straight lines. There is a considerable amount of distortion in this type of graph paper and the hydrologist should resist the false security that it may convey. In some cases, other types of graph paper are used for plotting. The probability scale is always distorted and, in some cases, both scales are distorted so that a certain probability distribution will plot as a straight line. In this manual, only lognormal paper will be used.

Flood probabilities will generally be stated not as probabilities at all, but rather as the recurrence interval (waiting time). This interval is the expected (average) time between the occurrence of values exceeding a given level. It can be shown that this interval is equal to one divided by the exceedance probability for that level. The value of this interval is expressed in terms of time. In hydrology, terms such as 25-, 50-, and 100-year floods are used, which refer to floods having annual exceedance probabilities of 0.04, 0.02 and 0.01, respectively. False meanings are often attached to floods that are referred to in terms of a waiting time, such as the 100-year flood. For example, a 100-year flood may be exceeded several times in 100 years, or it may not be exceeded at all during that time. The waiting time should not be confused by saying that once a 100-year flood has taken place, another 100-year flood cannot take place until 100 years have passed. Every year has an equal chance of having a 100-year flood, that chance being a probability of 0.01. If a 100-year flood occurred this year, the probability of another 100-year flood occurring next year is still the same 0.01. To rephrase, the chances of a 100-year flood are not altered by the flooding or nonflooding in other years.

*(c) **Probability Distributions.***—Many probability distributions are in common use; however, only a few of the distributions that are used in flood hydrology will be discussed. The discussion is oriented towards the concepts and elementary theory that support the use of these distributions. The applicability of the various distributions to flood hydrology will be discussed briefly.

In virtually all approaches used for flood frequency, the basic form or family of distributions is chosen prior to the analysis of the actual data. Once the basic form has been selected, the parameters of the distribution are estimated based on the sample data. An exception to this would be the use of a nonparametric approach, although this approach is rarely used. The choice for the basic form of the distribution can be based on the natural bounds assumed for the flood process, the theoretical considerations based on the assumed process and the behavior of statistics, and the statistical behavior that has been observed in data from other sites.

Zero flow is an obvious example of a lower bound for floods. For practical purposes, a lower bound equal to some type of pseudo-base flow may be reasonable to use. A distribution that preserves a lower bound would seem to be indicated. Also, if the PMF is considered to be an absolute upper bound, it is logical to assume the best choice for a flood distribution would also have the ability to preserve an upper bound. In actual practice, floods near zero or the lower bound are rarely of any interest, and whether the selected distribution preserves such a bound is of little concern. The same situation would also apply to the upper bound of the PMF; however, data do not exist to either verify or deny the existence of such a bound. In fact, there are not enough data to extend frequency curves to anywhere near this limit. Since the effects of this physical limit would probably not be felt until floods were quite near the limit, it is doubtful that anything would be gained through the use of distributions that explicitly recognize this limit.

In many natural processes, the random variable is the sum of several other variables or is the result of the influence of several factors, each of which has an additive effect on the variable. Statistical theory demonstrates conclusively that a random variable that is calculated as the sum of a number of other random variables that are not totally dependent upon one another will tend to be distributed according to the normal probability distribution. This conclusion governs many natural processes. For floods, the normal distribution is an obvious choice for cases where it can be argued that the influencing factors are several, are somewhat independent, are all of comparable weight or influence, and are additive. The normal distribution has only two parameters, the population mean and standard deviation.

If the factors are not additive but rather are multiplicative, the lognormal distribution can be argued to apply. In this case, the influence of the factors is in fact additive in terms of their logarithms, and the logarithms of the variable are normally distributed. The case for multiplicative factors can be argued for many types of flood flows because most flood data seem to behave in a lognormal manner. It should be noted that using this distribution preserves a zero flow lower bound. The two parameters are the population values of the mean of the logarithms and the standard deviation of the logarithms.

In some cases, a three-parameter lognormal distribution is used, where the logarithms of the values minus a constant are found to be normally distributed. The constant is a lower bound that is preserved by the distribution. The three parameters are the constant and the mean and standard deviation of the logarithms of the flows reduced by the constant. Other distributions can be formed as direct applications of the lognormal distribution to nonlinear transformations of the variable.

Another distribution that is a slight variation from the lognormal is the log-Pearson Type III. In addition to the mean and standard deviation of the logarithms, this distribution uses the coefficients of skew of the logarithms. For a zero skew, the distribution is exactly the lognormal distribution, or a straight line, when plotted on lognormal paper. When the coefficient of skew is positive, the curve is no longer a straight line, it is concave upward; and when the coefficient of skew is negative, the curve is concave downward. This form of distribution can easily be supported as a modification of the lognormal, where the assumptions for that distribution are not exactly met. This form of distribution has been found to fit a wide variety of flood flow data.

The "Gumbel" or "extreme-value" distribution, also known as the "double exponential" distribution, has gained some popularity for use in flood hydrology. This distribution is actually one of only three limiting extreme value distributions and, as such, has some theoretical justification for being used. In general, the assumptions and conditions required for the extreme value distribution are not satisfied with flood data and, while it does fit some data fairly well, this distribution is not widely used. The tails of this distribution behave much like the tails of the normal distribution.

A more flexible distribution known as the "generalized extreme value distribution" has also been proposed. This distribution combines all three extreme value distributions into one.

In some cases, it has been found that the upper or lower tail could be fit by a distribution but that both tails could not be fit at the same time. A distribution known as the "Wakeby", that has five parameters, has gained attention as a distribution that could handle this situation. The additional parameters allow for the increased flexibility needed to fit two seemingly incompatible tails.

(d) Parameter Estimation.—Several methods of parameter estimation are available, two of which will be discussed here. The "Method of Moments" [68] is probably the most commonly used parameter estimation method. In this method, the parameter estimates are related to the moments of the sample data. The first three sample moments used in hydrology are the mean, variance, and skew. Usually, the relationship is calculated from the mathematics of the basic distribution. In some cases, such as the normal distribution, the parameters of the distribution are actually population moments; in this case, the mean and variance. For this normal distribution, the parameter estimation is simply the use of the sample moments as estimates of the parameters (i.e., population moments). For other distributions, the parameters can be shown to be related to some function of the population moments. By using sample moments as estimates of the population moments and by using the indicated functions, the parameters are estimated as functions of the sample moments. This methodology for parameter estimation has the advantage of being straightforward and simple. The mathematics involved in deriving the functions is relatively basic, and the resulting functions are generally elementary. In addition, the sample moments are easily calculated and, in themselves, hold some inherent meaning for the practitioner. The mean is the first moment and is an indicator of central tendency or a location parameter. The variance is the second moment, or square of the standard deviation, and is an indication of the scale or spread of the distribution. The third moment is the expected value of the cubes of the deviations from the mean, and is usually stated in terms of the coefficient of skewness. This moment relates to the symmetry of the distribution. The fourth moment, often stated in terms of the kurtosis, can be interpreted in terms of either the peakedness of the distribution or the heaviness of the tails of the distribution. These four moments have a direct relationship to the location, slope, and shape of the plotted frequency curve.

The "Method of Maximum Likelihood" for parameter estimation is very appealing in that it appears to have a slight statistical advantage over the Method of Moments in the estimation of parameters for several types of distributions. Unlike the Method of Moments, it is easily adaptable to the use of historic and censored data. The method estimates the parameters such that the likelihood of experiencing the observed data is maximized. Unfortunately, the formulation procedure and application are more complex. Initially, this method involves evaluation of the likelihood of the sample data being observed. This is done in terms of unknown parameters that are still to be estimated, and the likelihood is usually expressed as a product. The probability density function is evaluated for each data value, and then all of the probability density values are multiplied together to get the likelihood function. The higher the value of the likelihood function, the higher the likelihood. To maximize the function with regard to the choice of the parameters, the partial derivatives are taken with respect to each of the parameters and then these derivatives are set equal to zero. This results in n equations, one for each partial derivative that has been set to zero, with n unknowns, the number of parameters. These equations are then solved for the parameter estimates.

7.4 Log-Pearson Type III Distribution

Because the log-Pearson Type III distribution is the principal distribution used by the Bureau, a complete section will be devoted to it. Karl Pearson [65] developed a series of distributions that have the ability to fit many different shapes of observed sample frequency distributions. The log-Pearson Type III distribution is an application of the Pearson Type III distribution applied to the logarithms of the data, and has been found to be very adaptable to floods and even more so to the logarithms of floods. The log-Pearson Type III distribution, hereafter called LP-III, when presented in the probability density form, usually has a bell shape or, with some parameters, a J-shape. Regardless of the shape of the curve, it is characterized by being noticeably skewed to the right. The application of LP-III is covered in detail in Bulletin 17B [14]. This section will not attempt to reproduce all of that information, but will provide supplemental information.

(a) Adoption by Federal Agencies.—In an attempt to achieve consistency in the Federal sector with regard to flood frequency studies, the U.S. Water Resources Council issued Bulletin 15, *A Uniform Technique for Determining Flood Flow Frequencies*" [11] in 1967. Later, Bulletin 15 was expanded and reissued as Bulletin 17, 17A, and, most recently in 1981, as 17B [12,13,14]. This bulletin was created as a basis to standardize flood frequency procedures within all Federal agencies, and is fully supported by the Bureau of Reclamation. This bulletin is now issued jointly by the various agencies under the auspices of the Interagency Advisory Committee on Water Data because the Water Resource Council no longer is in existence. The actual printing and distribution of this bulletin are currently handled by the Office of Water Data Collection of the USGS.

Considerable study and debate was expended prior to the selection of LP-III as the basic distribution to be used throughout the United States. The primary concern was in having a distribution that would be appropriate for use with data resulting from all types and causes of flooding. This was a pioneering effort that focused all flood frequency research for several decades. This effort came under much scrutiny and attack from both inside and outside the Federal sector. However, no better approach has been found and proven for general usage. It should be noted that Bulletin 17B allows other approaches to be used when justified; however, in most instances, the procedures described in this manual are sufficient and reliable.

(b) Form.—The LP-III distribution is a modification of the Pearson Type III distribution, where the logarithms of the flows are taken and then fit to the Pearson Type III distribution. The LP-III distribution has the following probability density form:

$$f(x) = p\left(1 + \frac{x}{a}\right)^{-cx/a} \tag{1}$$

where:

x = deviations of the variable from the mode, and
p, a, and c = parameters estimated from sample data.

The mode is the value most likely to occur. The distribution can also be written in a cumulative distribution function form that, in actual practice, is easier to use:

$$\log Q = M + KS \tag{2}$$

where:
Q = flow rate in cubic feet per second,
M = population mean of the logarithms of the flows,
K = a factor that is unique for each pair of values of exceedance probability and skew coefficient of the logarithms of the flows, and
S = population standard deviation of the logarithms of the flows.

The values for K in equation (2) are provided in Bulletin 17B[14] in an extensive table for a large range of combinations of probability and skew.

(c) ***Shape.***—The LP-III distribution is usually plotted on lognormal paper. If the coefficient of skew is zero, the distribution is lognormal and will plot as a straight line. If the skew is positive, it will be curved in a concave upward direction; if negative, it will curve concave downward. A high positive skew may be reason to subject the frequency analysis to further study. Normally, the flood data will have skew coefficients well below 1.0. Although Bulletin 17B [14] gives values of K for coefficients of skew up to 9.0, the use of values over 1.0 should be highly justified. A reasonable range of skew coefficients, as computed by procedures described in Bulletin 17B, is from −0.4 to +0.5. Values outside this range should be examined carefully and only used after thorough justification. Often, a high sample skew value is the result of an unrepresentative sample. Statistical analyses of randomly generated data samples, which were called the Monte Carlo experiments, demonstrated a high degree of unreliability in the estimation of the coefficient of skew, even with recorded periods over 100 years. There is negligible theoretic support and experience to justify the use of LP-III distributions that have high positive skew.

The effects of two or more independent causes for flooding, such as different storm types, can also be a reason for a high positive skew. If this is the case, the high positive skew is indeed proper for the population; however, it is recommended that the independent causes be separated and the population treated as a mixed population, as discussed later in this chapter.

Other reasons for a high positive skew include upstream flood control regulation, large amounts of upstream natural storage with constricted or regulated release, and trends in basin response. Generally, these effects need to be accounted for and the data adjusted to arrive at frequency values that can be used with confidence.

A high negative skew is also subject to suspect. Again, the cause may only be due to an unrepresentative sample. A high negative skew may be justified when the assumptions that tend to support a normal distribution are closer to being true than the assumptions for a lognormal distribution. For example, if the floods being analyzed are caused by many factors that are additive in their effects, the normal distribution may be expected. In this case, it probably would be justifiable to deviate from normal procedure and use normal distribution. However, if the LP-III is still used, the sample data may still fit reasonably well but a negative skew should be expected. Normally distributed data will plot on lognormal paper as negatively skewed. In this case, it would probably be best to plot the sample data on both lognormal and normal probability paper. Logical support and reasoning must be used and included in the flood study report to support the use of a highly skewed distribution. Unusual sample data alone cannot support acceptance of a frequency curve that defies common sense and logic.

Another theoretical justification for a negative skew would be the existence of an upper bound that would be reflected in the sample data as a tendency for the frequency curve to bend over as it approaches the bound. However, this is seldom found in the data due to the rare nature of any upper bound and the likelihood that boundary effects are only felt near the boundary. All evidence suggests that upper bounds must be much larger than, for example, the 100-year flood.

It should be noted that the LP-III distribution is usually a bounded distribution. Obviously, the flows cannot be below zero, and the fact that logarithms are used automatically sets a boundary. This distribution also usually has a boundary set on the logarithm of the flow. For zero skew, there is no bound; for negative skew, there is an upper bound; and for positive skew, there is a lower bound. The bounds for both positive and negative skews can easily be evaluated because they correspond to a value of $K = -2$ divided by the coefficient of skew. It should be noted that these bounds are not ordinarily of much influence or concern unless the skew values are highly negative or highly positive. However, if the absolute value of the coefficient of skew exceeds 1.0, or if resulting distribution is to be extrapolated beyond a probability of one divided by twice the length of record, the analyst should explicitly recognize the bound and justify the use of the bounded distribution.

(d) Advantages.—In addition to the advantage of offering consistency to Federal agencies, the LP-III distribution has several more advantages. As with any distribution that is adopted as a base, LP-III offers a reproducible analytic method in which the parameters involved are easily understood. The estimation procedure and method of moments are both well accepted and are also easily understood. Several computer programs are available to aid in the application to most situations. The distribution can also be easily applied without a computer program.

The LP-III distribution is almost the same as the lognormal distribution, which can be defended from a theoretical basis. By using a coefficient of skew other than zero, the distribution can be made to fit most data in a reasonably satisfying manner. In comprehensive studies, LP-III has been found to compete favorably with other distributions. It is also a versatile distribution that can closely fit data that varies from normal to lognormal to even heavier-tailed distributions.

*(e) **Disadvantages.***—Although the LP-III distribution has the advantage of closely fitting the data, that may have come from many different distributions, this may also be a disadvantage. One objective in selecting the basic distribution is that the behavior of the results are constrained, which means that the form of the distribution would restrict the results to be consistent with good overall judgment and to be consistent with the statistical behavior that has been found with previous data. In this manner, the choice of distribution would tend to compensate for the effects of unrepresentative samples and sampling variation. The LP-III distribution offers little in the way of beneficial constraint.

It can be argued that flood discharge values should not be expected to follow only one type of distribution, and that LP-III, because of its ability to fit data from many parent distributions, masks this behavior. The use of other more restrictive distributions, with the choice dictated by the circumstances of the application, may actually be a more technically efficient approach.

A major disadvantage of LP-III is the fact that it is highly dependent on the value of the adopted coefficient of skew. Sample estimates of this coefficient have been shown to be highly unreliable. This problem has been recognized in Bulletin 17B and mitigating procedures are recommended. This bulletin encourages the use of a generalized skew coefficient and using a weighted average of the generalized and station (sample) values of the skew coefficient.

As with any fitted distribution, the results are still only an estimate of the population distribution and are prone to errors due to errors in basic data, random sample variation, and the effect of an unrepresentative sample including the effects of outliers. In addition, the results are subject to very real errors with respect to the validity of the underlying assumptions that have been made. If the true population distribution is not LP-III, then an additional error has been introduced. Statistics does offer some help in evaluating the reliability of the results. Confidence bands are a highly recommended approach for this type of evaluation and should be presented in the flood study report.

*(f) **Fitting Procedure.***—The current fitting procedure used in Bulletin 17B is the "method of moments" estimation. This method uses the logarithms of the flows and, in reality, is a fitting of the LP-III distribution to the logarithms of the flows. The mean, standard deviation, and coefficient of skew are estimated from the sample data, all in terms of logarithms. The mean value is indicative of the basin size and the basic runoff regime. This value differs from basin to basin and from one geographic area to another; however, basin size has only negligible correlation with standard deviation. The coefficient of skew is assumed to be

reasonably stable; however, because of the extreme sampling variation experienced with this parameter, this assumption is not easily shown using actual data. The use of maps that relate coefficient of skew to location are an indication of the philosophy that the values for this parameter should be expected to vary in a rather gradual manner with location, and be mostly independent of the basin characteristics. Again, it is very difficult to support or deny this philosophy with actual experience.

(g) Generalized Skew.—Throughout much of the literature, the term "skew" is used when the term "coefficient of skew" is the one intended. This same nomenclature is used in this chapter. Generalized skew is a skew value that reflects the basic philosophy that the true population values for skew are relatively stable and, while not necessarily constant, vary in a gradual manner with primarily location and secondarily with other factors.

There are three approaches in Bulletin 17B for the estimation of a generalized skew. One approach is the use of maps that have contour lines showing equal values of skew. These iso-skew line maps are based on the analysis of many individual stations. The reliability of the map values is estimated as a constant throughout the United States based on how well the contour lines match with the data points for the individual stations. These maps only reflect the influence of locations on the values of skew, and reflect an assumption that basin size, cover, slope, elevation, drainage density, and other factors are negligible. Users may develop equivalent maps for the area around the basin of interest. This approach seems to produce somewhat reasonable results, but care must be taken not to cover up the effects of mixed populations, which do depend on drainage area, by using this type of smoothed value. It should be noted that the skew maps in Bulletin 17B were developed without taking into account any effects of mixed population.

A second approach used for estimating a generalized skew is a prediction equation approach. In this case, skew values are estimated for many basins in the same hydrologically and meteorologically homogeneous area. Factors believed to be potentially important would also be estimated for each area. These estimated values are then related to the skew values by a fitted prediction equation. The Bureau's general experience with this approach is that the relationships are weak and prone to producing questionable results. If this approach is used, care must be taken to avoid spurious results, ensure that prediction equation is indeed statistically significant, and that the equation is meaningful and consistent with hydrologic common sense.

A third approach, and the one most often used by the Bureau, is a weighted average of the skew estimates from several basins in a hydrologically and meteorologically similar area. The most commonly used weight is the number of years of record. Weights could also be based on the statistical reliability of the individual estimates, but this is nearly the same as using the number of years of record. It is recommended that the weighted average of the station skew and the generalized skew be calculated. The weights are based on the estimated reliability of the two estimates. It is this weighted average value of skew that is to be adopted as the parameter estimate.

The regionalization inherent in the generalized skew and the averaging of the generalized skew with the sample estimate reflects a general lack of confidence in the sample skew estimates. These procedures are not used for the mean and standard deviation parameters because their estimates are much more stable.

(h) Outlier Criteria.—Bulletin 17B includes a procedure for the identification of outliers. It is important to note that this procedure does not call for the automatic elimination of these data; however, it does call the analyst's attention to the need for further study and the possibility that the outliers may not be representative, and that some adjustment may be necessary. The procedure should definitely not be taken to mean that the data is in error. It should also be noted that the presence of a high outlier may well indicate that the data are comprised of a mixed population.

The outlier test is based on the assumption that the data fit the lognormal distribution and the sample mean and standard deviation are used. The test is performed at the 10-percent level (one sided). Practitioners should be cautious when throwing out data points because elimination of high values from the data set will result in a general downward bias in flood frequency results. Bureau practice is that high outliers be treated as historical data rather than being eliminated. Low outliers are generally not of concern in a flood frequency analysis; however, the effects on the upper portion of the frequency curve should dictate the elimination, modification, or retention of them.

(i) Adjustments.—Bulletin 17B includes many adjustment procedures for such conditions as zero flows and incomplete records common in the Western States. These procedures are used in Bureau flood frequency studies with the following two exceptions:

Exception 1.—The expected probability adjustment is generally not used in Bureau flood frequency analyses. This adjustment can be justified when evaluating flood damages or performing other economic evaluations. Since economic analyses for flood control are not usually performed by the Bureau, this adjustment is not applied.

Exception 2.—The historic data adjustment has been conclusively shown not to perform well with paleohydrology data or other data that represent a much longer time period than that of the systematic record. For these cases, it is recommended that this adjustment not be used.

7.5 Example Application Using Bulletin 17B

It is not practical to reproduce the entire contents of Bulletin 17B in this manual even though any example would be incomplete without the necessary tables and other material contained in this bulletin. However, brief sample problems are shown in sections 7.11 and 7.12 to indicate how a mixed population data set is analyzed and how to compute confidence limits.

7.6 Limitations on Frequency Curve Extrapolation

An ultimate goal would be to arrive at a frequency curve that is valid over the entire range of possible flood flows. This of course is not possible because sufficient data do not exist to verify the choice of the base distribution. The sample data is only sufficient to provide estimates for the distribution's parameters. The errors that are unavoidable in the parameter estimates become intolerable once the frequency curve is extrapolated beyond a certain point.

The reliability of the frequency curve gradually decreases for the more extreme events. For more common events, the reliability of the frequency curve is fairly good, but as events are estimated that have return periods comparable to or longer than the sample length, the reliability becomes poor. This reliability deterioration does not take place at a particular point, it initially deteriorates slowly and takes place in a gradual manner. As the exceedance frequency becomes smaller, the rate of deterioration increases. In addition to the deterioration in the quality of estimates due to sampling errors, the reliability of the basic choice of distribution becomes questionable. For example, it is difficult to defend the tail behavior of a distribution for a 200- or 300-year return interval when the basis for choosing the distribution are records rarely longer than 60 years.

The sample length is the main controlling factor concerning the reliability of the frequency curve. It should be noted that the use for which the curve is to be put is also a consideration. If choices concerning human life and dam safety are involved, more caution should be taken about using extrapolated frequency curves than if the use was for determining the size of a cross drainage structure where only a short term loss of service is the main consequence of failure.

Practical rule-of-thumb knowledge, which is supported by statistical calculations, indicates that frequency curves are reasonably reliable out to return periods of about the sample record length, or even twice the sample length. The current Bureau practice is to limit the extrapolation of the curves to twice the length of record, or 100 years, whichever is longer. In cases where catastrophic loss, loss of life, or dam safety are not involved, further extrapolations can be used as justified on a case-by-case basis.

Confidence bands are used as an excellent approach to quantifying the uncertainty in the frequency curve estimates. Within the Bureau, confidence bands are constructed at the 5- and 95-percent levels, and are generally extrapolated twice as far as the frequency extrapolation curve; that is, 200 years or four times the length of record, whichever is longer. The length of record is needed to calculate the confidence bands. Due to the nature of this calculation, the sample length is used whenever the gauged data is at the site and also for data transposed from another site. When using regional approaches, the typical or average record length is used. When precipitation frequency data are used with rainfall-runoff modeling to generate flood data, an assumed length of record of 20 years is used.

7.7 Mixed Populations

In most flood frequency analyses, the data are assumed to come from one basic distribution; however, it is frequently found that the floods are the result of distinctly different causative factors. This phenomenon was recognized by Allen Hazen in his 1930 publication, *Flood Flows* [64]. In these cases, the population follows a distribution that is a combination of two or more base distributions. Often, while the existence of a mixed population is recognized, the analysis is conducted as an expediency without taking this into account. However, it is generally considered that improved results are obtained with an analysis that specifically treats the mixed population. This is especially true in the Western States where data reflecting runoff from snowmelt and rainfall are quite common.

(a) Causative Meteorological Factors.—The usual cause of a mixed population is the existence of flooding due to different types of storms. Different storms are typically snowstorms that lead to the accumulation of a snowpack (with resulting snowmelt flooding), general storms of low intensity and long duration, local storms of short duration and high intensity, and hurricanes that produce intense rainfall over large areas.

The typical frequency curve for snowmelt runoff is usually very flat; that is, it is characterized by a low standard deviation of the logarithms. Usually, the curve is negatively skewed and, with large basins, is more nearly normally distributed rather than lognormally.

The frequency curve for floods resulting from general type storms is steeper (has a relatively higher standard deviation) than the snowmelt curve. The lognormal distribution is a reasonably typical distribution for general storm-produced floods. In most locations in the mountainous regions of the West, the general storm-produced flooding will dominate the rarer floods while the more common floods are predominately snowmelt. As the basin location changes, the relative intersection of the curves also changes. The intersection tends to move towards rarer floods as the basin elevation increases. Either type of flooding may tend to overshadow the other for the entire set of data.

As with floods resulting from general storms, floods resulting from local storms are closely approximated by the lognormal distribution. However, local and general storms are distinctly different as are their flood frequency curves. The frequency curve for floods resulting from local type storms is much steeper than the general storm curve and, as a result, local storms tend to dominate a less frequent portion of the frequency curve.

The hurricane frequency curve is the steepest of the curves discussed. These hurricane type floods are much rarer than the others, most years will not even have a flood of this type. The magnitude of the flooding is highly variable, reflected by a typically large standard deviation.

(b) Types of Mixed Populations.—Although a mixed population condition is probably due to different meteorological factors, a mixture could also be the result of distinctly different hydrological factors. These factors might include differences in infiltration, cover, channel roughness, and antecedent conditions, with the differences possibly related to time of year. Usually, a hydrologic cause for mixed population behavior is not nearly as clear, and may tend to be unidentifiable. Hydrologic differences may help explain the existence of a slightly positive skewed frequency curve in the absence of mixed meteorlogical causes.

(c) Separation of Data.—To analyze a mixed population, the data must be segregated by meteorological cause, and it is best to develop a separate annual series for each cause. The data resulting from each cause is then analyzed separately with the results later combined to form a single curve. Often, rather than identifying the meteorological cause of each flood, the time of year is taken as an expedient method to achieve almost the same results. The situation is more complex when different meteorological causes tend to blend together, with a single flood being the combined result of two or more causes. An example of this would be a rain-on-snow flood.

(d) Combined Frequency Curves.—The final or combined frequency curve is a simple combination of the individual frequency curves using basic statistical principles. The probability of exceeding any given level of flooding is the sum of all of the individual exceedance probabilities minus a correction for the possibilities of having more than one of the causes producing a flood exceeding this level.

In the case of two causes, the probability of having a flood exceed a given level is equal to the sum of the probability of the first flood type and the probability of the second flood type, minus the probability of having both types of floods in the same year. The probability of having both floods in the same year is simply equal to the product of the two individual probabilities. Naturally, this joint probability is based on the assumption that the two types of flooding are independent.

When more than two causes are present, the same approach may be repeated. Equations are easily derived for any number of casual factors. For two causes, the equation is:

$$Pt(Q > x) = Pa(Q > x) + Pb(Q > x) - Pa(Q > x)\,Pb(Q > x), \text{ or}$$

$$Pt = Pa + Pb - Pa\,Pb \tag{3}$$

where:

Pt = total curve value, and
Pa and Pb = individual exceedance probabilities.

For three causes, the equation is:

$$Pt = Pa + Pb + Pc - Pa\,Pb - Pa\,Pc - Pb\,Pc + Pa\,Pb\,Pc \tag{4}$$

Figure 7-4 illustrates the case with two casual factors. Note that the data points and confidence bands are not shown, only the individual curves and the resulting total curve. The resulting total curve is typical of what might be seen in the actual data (not separated) for the site. The total curve shows two distinct segments that, for this case, are straight lines on lognormal paper with a rather sharp break. This behavior is typical.

Frequently, one casual factor may not be important except for rather rare floods, and such a flood may be so rare that the recorded data is entirely dominated by a more common type of flooding. After extrapolation, the rare type of flooding may become dominate. In this situation, a mixed population analysis is even more desirable. This situation has been shown to be common in the higher elevations of the foothills in Colorado, where the mixture is somewhere between snowmelt floods and rainfloods.

7.8 Volume Analysis

Although most flood frequency analyses are performed using terms of instantaneous peak flows, a complete hydrograph is often needed. In the operation of a dam, the volume of water is sometimes as critical, if not more critical, than the peak flow. Criteria provided in Bulletin 17B [14] are intended for peak analysis and must be used with caution when applied to volume data. Some of this caution is due to the fact that the

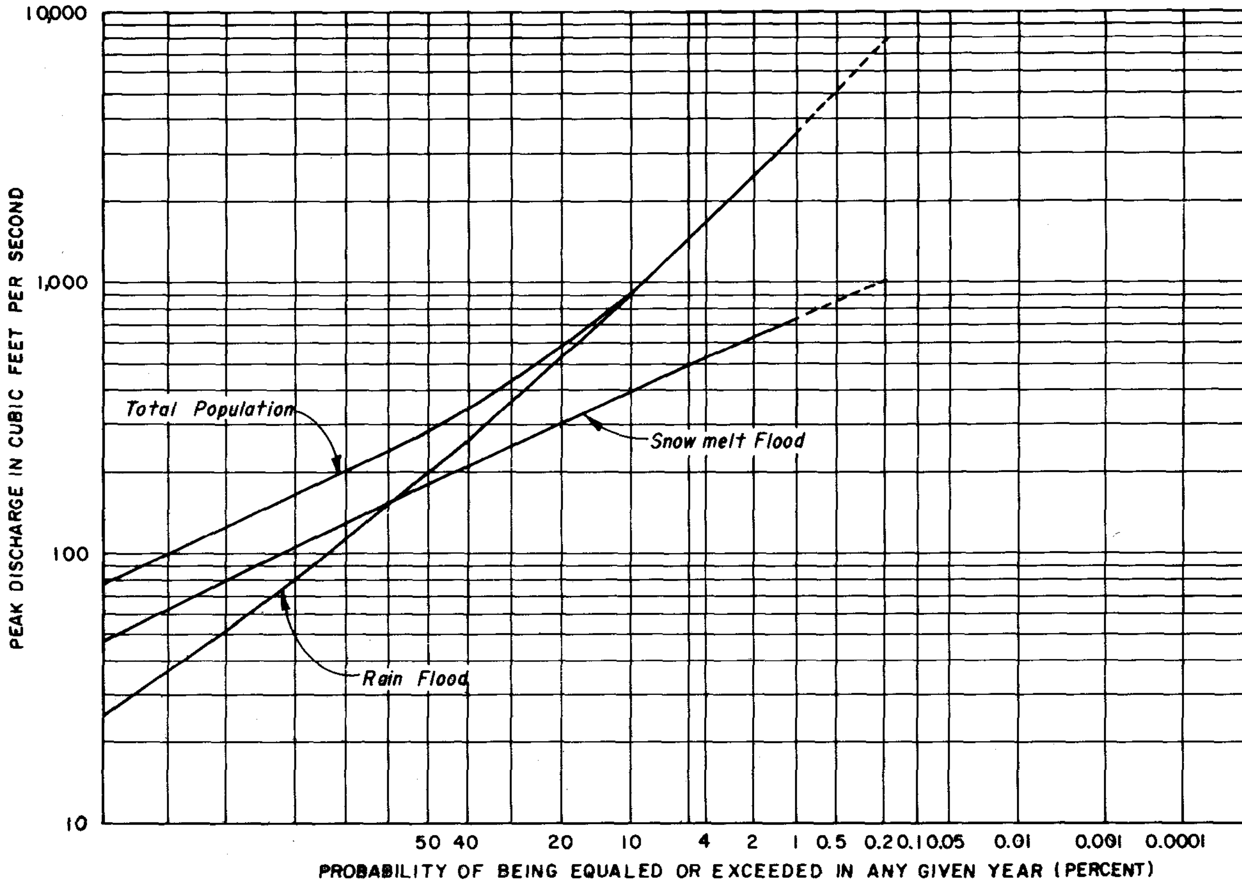


Figure 7-4.—Mixed population frequency curve. North Fork of Big Thompson River near Drake, Colorado. 103-D-1936.

parent or population distribution form is expected to change as the unit time changes from instantaneous to daily, to 3 days, to 15 days, etc. The storm types that define the frequency curve may even vary as the unit time changes.

*(a) **Balanced Flood Hydrograph.***—For a particular flood study, it may be obvious that volume is very critical. What is not obvious is the unit time that is most critical for a given return period. Generally, the desire is to afford a certain degree of protection, and that may be the 1 in 100 or 100-year level. In this case, it is desirable to provide protection against all durations of floods that have that exceedance probability. One approach would be to evaluate the volumes of floods of all unit durations and to fit reasonably shaped hydrographs to those volumes. Each of these hydrographs would then be tested to determine which was most critical. A second, and most used, approach would be to fit a single hydrograph to all of the volume data for the various unit times. This approach is known as the "balanced hydrograph technique." This approach results in only one hydrograph to be tested.

To construct a balanced hydrograph, a symmetrical or other reasonable shape is usually selected as a guide for the fitting of the volumes. The instantaneous peak is initially fit, then the peak daily value, followed by the next shortest unit time volume, etc. In this manner, a hydrograph is created that provides a constant level of probability regardless of the unit time selected. By routing such a hydrograph through a dam, the routing process actually selects the most critical unit time. Using the balanced hydrograph, the most critical water level will be very close to the most critical water level determined by using separate hydrographs that have been fit to each of the different unit time volume values. In effect, the balanced hydrograph approach is an expediency that allows for a fairly complete analysis with a fraction of the effort and produces somewhat conservative (high side) results.

In the construction of a balanced hydrograph, be aware that different flood types may actually be mixed within the hydrograph. The storm expected to result in the 100-year peak flow may not be the same storm expected to produce the 100-year, 7-day volume. Also, it would not be expected that the 100-year peak flow and 100-year volume would be experienced in the same flood. While this procedure is not conceptually pleasing, it must be remembered that this approach is an expediency and not a model of reality.

It is not uncommon to have inconsistencies between the volume frequency values for the different unit times. For example, the 100-year volumes are usually extrapolated values, and it is possible to actually estimate frequency curves that have a higher 15-day volume than 30-day volume at the 100-year return period. Obviously, this is a physical impossibility; however, it is not uncommon, especially if the values of skew vary considerably between unit times. This type of conflict must be resolved in a reasonable manner. The most common approach would be to achieve some smoothing of the skew coefficients with respect to unit time. This type of problem is usually only found at the longer unit times, and the inconsistency usually amounts to only an insignificant volume.

(b) Fixed Interval Analysis.—One source of potential problems in the construction and use of balanced hydrographs is what is often called "fixed interval analysis." For example, in the construction of the balanced hydrograph, the peak 1-day volume is used. This value is determined from daily flow values, not peak 24-hour values. Note that peak daily values are constrained to start at midnight, thus the term "fixed interval," while the peak 24-hour values start at any time of day. A peak 24-hour value for 1 year will always exceed the peak daily value. When constructing the balanced hydrograph, the peak daily flow value is usually used as being the same as the peak 24-hour value. As a result, the balanced hydrograph has slightly less volume near the peak. This problem is considered to be of minor concern, but some awareness is justified.

A more serious fixed interval problem exists if monthly data are taken to be 30-day volume data. Again, the use of fixed interval data will result in low estimates, and care should be taken to quantify the magnitude of this problem. The decision as to whether the fixed interval data is adequate will depend on how critical the volume frequency data is to the study at hand.

7.9 Probability Relationships for Ungauged Areas

Usually, the site where the frequency analysis is needed is not a site of recorded flow data, and the site is said to be an "ungauged area." Three basic approaches are used to estimate a frequency curve for such an ungauged site: (1) transposition of frequency data, (2) regional flood frequency, and (3) use of synthetic data.

(a) Transposition of Frequency Data.—The transfer of information from one site to another is required for any ungauged area. A basic relationship found to work well is that the ratio of flows at two locations are assumed equal to the ratio of the square root of the drainage areas, or:

$$\frac{Q_1}{Q_2} = \frac{A_1^{0.5}}{A_2^{0.5}} \qquad (5)$$

This square root relationship has been found to be fairly accurate for instantaneous peaks and for the short duration volume frequency values. For longer duration volumes of about 60 days or more, a higher exponential power of the ratio of the drainage areas may be more applicable, with the power increasing from 0.5 to 0.8 as the unit time is increased.

For annual runoff, this power may approach 1.0. It should be noted that this approach does not take into account any differences in basin factors other than drainage area alone. In effect, the LP-III frequency curve being transposed is only changed in terms of the mean; the standard deviation and coefficient of skew are not altered.

It is preferable to transpose information from one site to another on the same stream rather than from one stream to another. Lacking this preference, it is preferable to transfer information between basins that are hydrologically similar. Small changes in basin size are also preferable. Also, some attention should be given to the similarities and differences between the sources of floods within the basins.

(b) Regional Flood Frequency.—There are three basic approaches to regional flood frequency: (1) average parameter approach, (2) specific frequency flood approach, and (3) index flood approach. The depth and scope of these three approaches vary considerably; however, similar results are obtained from all three.

The average parameter approach uses data at many similar sites in a homogeneous region to estimate an average value for a parameter. In many applications, the parameter is not really an average but rather a function of basin factors. In effect, the value for the parameter is an average value for that basin's size and characteristics. The most common average parameter approach used is to assume that the mean value parameter (logarithms) varies with the drainage area, while the standard deviation and skew coefficient do not. Generally, the log mean of the flows is plotted against drainage area and a graphical relationship developed. The next most common type of parameter approach would be to relate the mean not only to area but also to other factors such as location, annual precipitation, basin elevation, and cover. Another more complex and most often not justifiable type of average parameter approach is to also relate the standard deviation to basin factors. The coefficient of skew could also be related to other factors (generalized skew) but, for an ungauged area, this refinement often cannot be justified.

The specific frequency flood approach begins by analyzing the flood frequency at many stations in the region. Then, a relationship is developed that relates the values for a single return period, such as the 100-year values, to the basin areas and other basin factors. Obviously, this approach produces results similar to those obtained using the average parameter approach.

The index flood approach usually requires the use of concurrent data from several stations, with the data from each station being scaled using the mean annual flood or some similar value. In some cases, the drainage area (square root) might be used to scale the data. The resulting scaled data are then combined as if all the data came from a single site. When all of the stations are concurrent, this approach is equivalent to the average parameter approach where the mean is related to the drainage area. The USGS has developed several guides for determining regional flood frequency estimates that are usually done on a State-by-State basis with a State often divided into several regions.

(c) Frequency Analysis From Synthetic Data.—Rainfall runoff models have been used for developing flood frequency curves by using precipitation data. The model must include the ability to model soil-water retention and loss to produce reliable results. Also, the model must be run for a considerable time with concurrent inputs and measured flow data to calibrate the model. To achieve reasonable initial soil moisture conditions at the time of major flood producing storms, the model must run continuously.

An alternative approach to using actual data would be to make use of stochastic rainfall and, possibly, stochastic soil conditions. This approach is usually not justified because of the added work involved. Since this approach is still being developed, it is not considered to be practical for most studies at this time.

7.10 General Trends in Frequency Relationships

Experience has shown several identifiable trends in frequency relationships. As a single basin parameter changes, with other parameters held constant, the parameters of the LP-III distribution can be expected to change in a logical manner. Obviously, it is not practical to find sites where only one basin parameter changes because many parameters change simultaneously. The basin parameters are often interrelated, and the effects of changes in two parameters may be either complimentary or contradictory. Care must be taken to recognize not only the isolated effects of individual basin parameters, but also the complex interactions between them. As an example, the 100-year peak flow would be expected to increase with increasing basin size, and to decrease with increasing channel length if the basin size and all other factors are held constant. However, since area and channel length are strongly correlated, experience shows that the values for the flood increase with channel length when that factor is examined alone. If both factors are incorporated into one relationship, the flood values would be expected to be directly related to the area while inversely related to the length. This relationship might be of the form of a power relationship where the basin area would have a positive power while the length would have a negative power. Some of the more important basin factors will be discussed in this section with respect as to how they typically influence the frequency distribution. These discussions are based on what has typically been found in actual practice.

The drainage area is the strongest single factor, and a strong almost linear relationship is found between the log of the area and the log mean of the LP-III distribution. Usually, this relationship is on the order of the flows being directly related to the square root (a power of 0.5) of the drainage area. With volume floods, as unit time increases, the power would be expected to increase. Note that basin slope and channel lengths all change with changes in basin area; these observations are with typical concurrent changes in these factors. The log standard deviation is not strongly related to basin area. The log coefficient of skew can be expected to change in an inverse relationship with area; however, this is not a strong relationship. With large increases in area, the distribution can be expected to change slowly from lognormal to normal distribution, which results in a reduction in the log coefficient of skew. Sampling variation is high in the case of skew and, for this reason, relationships are not distinct.

As the elevation increases, all three parameters of the LP-III distribution decrease. This relationship is true even when the basin size is held constant.

Mixed populations result in an inflated log skew when the data is analyzed as a single population. This is a direct result of the mathematics. If the more common flood subpopulation is taken as the base, the entire population will have a small increase in the log mean, a larger increase in log standard deviation, and an even larger increase in log skew.

With a change in location, the effects are mainly related to the distance from the moisture sources; climatic effects on the flood type; e.g., snowmelt becomes more important in colder climates; and effects on other basin factors. Basin factors such as elevation, slope, soil, and cover all tend to change in a somewhat consistent pattern with a change in location.

For an increase in the unit time, the log mean usually increases while the log standard deviation and log skew decrease slightly.

If the effects of basin channel slope or channel length are examined, the effect of basin size must be examined simultaneously or spurious relationships may be developed, and many relationships are possible. The mean flow, not the log mean flow, is closely related to the square root of the drainage area, to the product of the drainage area and slope, or to the drainage area divided by the length, possibly to a power slightly less than one. Slope is typically closely related to one divided by the square root of the drainage area, while channel length and basin channel slope are inversely related.

7.11 Foss Dam Example

This section continues the example on the drainage basin above Foss Dam in Oklahoma. The derivation of the concurrent PMF hydrographs was illustrated in section 4.5 of chapter 4, and the routing and combining of these hydrographs was discussed in section 5.8 of chapter 5. Since Foss Dam is an existing dam for which modifications are assumed to be required, a discharge probability relationship, including confidence bands, is required to provide for the care and diversion of flows during construction of the modifications.

A stream gauge, operated by the USGS, has recorded flows at a location on the Washita River near Cheyenne, Oklahoma since 1938. The available annual peak discharge record extends from 1938 to 1984, 47 years of record. This gauge records flows from 792 square miles of the total 1,466-square mile drainage area above Foss Dam. Since the data reflect runoff from only part of the basin above the point of interest, the discharge-probability relationship at the gauging station will require transposition to the point of interest. Table 7-1 shows the annual peak discharges for each year of record along with other information necessary for determining the discharge-probability relationship.

Table 7-1 shows that the number of items of data N is 47, which is used in both the determination of the Weibull plotting position and in the development of the analytical discharge-probability relationship. The mean of the logarithms of the annual peak discharges $\overline{X}$ is:

$$\overline{X} = \Sigma X/N = 156.87558/47 = 3.33778, \text{ or } 2{,}176 \text{ ft}^3/\text{s}$$

The standard deviation S of the logarithms of the annual discharges is:

$$S = \left(\frac{\Sigma X^2-(\Sigma X)^2/N}{N-1}\right)^{0.5} = \left(\frac{543.222-(156.87558)^2/47}{47-1}\right)^{0.5} = 0.6528$$

The skew coefficient G_1 of the logarithms of the annual peak discharges is:

$$G_1 = \frac{N^2(\Sigma X^3)-3N(\Sigma X)(\Sigma X^2)+(2)(\Sigma X)^3}{N(N-1)(N-2)(S^3)}$$

$$= \frac{(47)^2(1940.4225)-(3)(47)(156.87558)(543.222)+(2)(156.87558)^3}{(47)(47-1)(47-2)(0.6528)^3}$$

$$= -0.2949$$

From the previous computations, it has been determined that, for the gauging station's annual peak discharges, the mean of their logarithms $\overline{X} = 3.3378$, standard deviation of logarithms $S = 0.6528$, and skew coefficient of logarithms $G_1 = -0.2949$.

Bulletin 17B recommends weighting the skew coefficient with the generalized skew coefficient given in the bulletin. For the station in this example, the bulletin's generalized skew coefficient for the logarithms

Table 7-1.—Annual peak discharges for each year of record for the drainage area above Foss Dam.

(1) Year	(2) Annual Peak Discharge, ft³/s	(3) Ranked Annual Peak Discharge, ft³/s	(4) Weibull Plotting Position	(5) Logarithm of Discharge X	(6) X^2	(7) X^3
1938	14,600	69,800	0.021	4.84386	23.46298	113.65139
1939	3,070	40,000	.042	4.60206	21.17896	97.46683
1940	1,080	14,600	.063	4.16435	17.34181	72.21737
1941	40,000	14,000	.083	4.14613	17.19039	71.27359
1942	14,000	11,900	.104	4.07555	16.61011	67.69533
1943	2,190	9,900	.125	3.99563	15.96506	63.79047
1944	1,240	8,900	.146	3.94939	15.59768	61.60133
1945	9,900	8,900	.167	3.94939	15.59768	61.60133
1946	8,900	8,450	.189	3.92686	15.42023	60.55308
1947	7,100	7,310	.208	3.86392	14.92988	57.68785
1948	8,900	7,100	.229	3.85126	14.83220	57.12267
1949	11,900	6,420	.250	3.80754	14.49736	55.19928
1950	8,450	5,830	.271	3.76567	14.18027	53.39822
1951	5,040	5,040	.292	3,70243	13.70799	50.75287
1952	465	4,710	.313	3.67302	13.49108	49.55299
1953	3,550	4,660	.333	3.66839	13.45709	49.36584
1954	69,800	4,470	.354	3.65031	13.32476	48.63952
1955	5,830	4,210	.375	3.62428	13.13541	47.60639
1956	3,890	3,890	.396	3.58995	12.88774	46.26635
1957	4,210	3,550	.417	3.55023	12.60413	44.74757
1958	1,750	3,070	.438	3.48714	12.16015	42.40413
1959	6,420	2,990	.458	3.47567	12.08028	41.98707
1960	1,510	2,930	.479	3.46687	12.01919	41.66896
1961	7,310	2,280	.500	3.35793	11.27569	37.86299
1962	2,930	2,190	.521	3.34044	11.15854	37.27443
1963	574	1,960	.542	3.29226	10.83898	35.68473
1964	159	1,800	.563	3.25527	10.59678	34.49539
1965	1,400	1,750	.583	3.24304	10.51731	34.10805
1966	1,800	1,510	.604	3.17898	10.10591	32.12650
1967	2,990	1,420	.625	3.15229	9.93693	31.32409
1968	4,470	1,400	.646	3.14613	9.89813	31.14082
1969	2,280	1,360	.667	3.13354	9.81907	30.76846
1970	734	1,240	.688	3.09342	9.56925	29.60170
1971	4,710	1,080	.708	3.03342	9.20164	27.91243
1972	1,360	1,050	.729	3.02119	9.12759	27.57618
1973	265	734	.750	2.86570	8.21224	23.53381
1974	592	592	.771	2.77232	7.68576	21.30738
1975	1,050	574	.792	2.75891	7.61158	20.99968
1976	1,960	560	.813	2.74819	7.55255	20.75584
1977	4,660	465	.833	2.66745	7.11529	18.97968
1978	297	427	.854	2.63043	6.91916	18.20037
1979	400	400	.875	2.60206	6.77072	17.61781
1980	560	297	.896	2.47276	6.11454	15.11980
1981	38	265	.917	2.42325	5.87214	14.22967
1982	1,420	159	.938	2.20140	4.84616	10.66834
1983	427	119	.958	2.07555	4.30791	8.94128
1984	119	38	.979	1.57978	2.49570	3.94266
			Totals	156.87558	543.2220	1940.4225

of annual peak discharges is −0.150. The following equations from Bulletin 17B are used to estimate the weighted skew coefficient G_W:

$$G_W = \frac{MSE_{\overline{G}}\,(G) + MSE_G\,(\overline{G})}{MSE_{\overline{G}} + MSE_G} \tag{6}$$

where:
G_W = weighted skew coefficient,
$MSE_{\overline{G}}$ = mean-square error of generalized skew,
MSE_G = mean-square error of gauging station skew,
G = computed gauging station skew, and
$\overline{G}$ = generalized skew from Bulletin 17B[14].

The value of $MSE_{\overline{G}}$ in equation (6) is always 0.302, and the value of MSE_G is found by applying the following equation:

$$MSE_G \approx 10 \exp \{A - B[\log_{10}(N/10)]\} \tag{7}$$

where:
$A = -0.33 + 0.08\,|G|$, if $|G| \leq 0.90$
$ = -0.52 + 0.30\,|G|$, if $|G| > 0.90$
$B = 0.94 - 0.26\,|G|$, if $|G| \leq 1.50$
$ = 0.55$, if $|G| > 1.50$

The computed gauging station skew G was previously found to be −0.2949; therefore, $A = -0.3064$ and $B = 0.8633$. Entering these values into equation (7) results in:

$$MSE_G = 10 \exp -0.3064 - 0.8633\,[\log_{10}(47/10)] = 0.1299$$

Then, by using equation (6), solve for G_W:

$$G_W = \frac{0.302(-0.2949) + 0.1299\,(-0.150)}{0.302 + 0.1299} = -0.251$$

The required information is now available to fit the LP-III distribution to the recorded annual peak discharges using the equation:

$$\log Q = \overline{X} + KS \tag{8}$$

where:
$\log Q$ = logarithm of discharge Q at selected exceedence probability,
$\overline{X}$ = mean of logarithms of annual peak discharges,
K = a factor that is a function of the weighted skew coefficient G_W and selected exceedence probability, and
S = standard deviation of annual peak discharge logarithms.

The development of the data for constructing the discharge-probability relationship at the gauge is shown in table 7-2. The values for both the actual observed annual peak discharges and their respective plotting positions, and the analytically derived discharges and their respective exceedence probabilities are plotted on lognormal probability paper, as shown on figure 7-4. This figure depicts the discharge probability relationship for the stream gauge on the Washita River near Cheyenne, Oklahoma.

The next step is to develop the confidence limits. Bureau practice is to display confidence bands at the 5- and 95-percent levels as an indicator of the inherent hydrologic uncertainties in discharge-probability relationships. The equations for computing points that define the upper and lower confidence band levels are:

$$Q_P^U = \overline{X} + K_P^U (S) \tag{9}$$

where:

Q_P^U = logarithm of discharge on upper confidence band curve U at annual probability P,
$\overline{X}$ = mean of logarithms of annual peak discharges,
K_P^U = value of K at probability P that defines upper confidence band curve U, and
S = standard deviation of annual peak discharge logarithms.

$$Q_P^L = \overline{X} + K_P^L (S) \tag{10}$$

where:

Q_P^L = logarithm of discharge on lower confidence band curve L at annual probability P,
K_P^L = value of K at probability P that defines lower confidence band curve L, and
$\overline{X}$ and S = as previously defined in equation (9).

Since $\overline{X}$ and S have previously been computed, it remains to compute K_P^U and K_P^L, which is accomplished using the following equations and procedures:

$$K_P^U = \frac{K_{G_W,P} + (K^2_{G_W,P} - ab)^{0.5}}{a} \tag{11}$$

where:

K_P^U = as previously defined in equation (9);
$K_{G_W,P}$ = LP III deviates, values for K_{G_W} are shown in table 7-3 for different values of P;
$a = 1 - \left(\frac{Z_c^2}{(N-1)}\right)$, where Z_c = 1.64485 for the 5- and 95-percent confidence band levels, and N = number of years of record; and

Table 7-2.—Development of data for construction of discharge-probability relationship. Washita River near Cheyenne, Oklahoma.

(1) Annual Exceedence Probability	(2) Return Period, years	(3) $\bar{X}$	(4) S	(5) K[1]	(6) KS	(7) log Q, (3) + (6)	(8) Discharge[2], ft³/s
0.5	2	3.3378	0.6528	0.04159	0.02715	3.36495	2,317
.2	5	3.3378	.6528	0.85136	0.55577	3.89357	7,827
.1	10	3.3378	.6528	1.25170	0.81711	4.15491	14,286
.04	25	3.3378	.6528	1.66164	1.08472	4.42252	26,456
.02	50	3.3378	.6528	1.91729	1.25161	4.58941	38,851
.01	100	3.3378	.6528	2.14117	1.39776	4.73556	54,395

[1] Interpolated from the values −0.2 and −0.3 given in Bulletin 17B[14].

[2] Discharges shown are to nearest cubic foot per second (ft³/s) to facilitate reader's understanding. Normally, these numbers would be rounded to nearest 10 ft³/s for values up to 10,000 ft³/s and to nearest 100 ft³/s for values over 10,000 ft³/s.

Table 7-3.—The 5- and 95-percent confidence band calculations for the Washita River near Cheyenne, Oklahoma.

(1) Annual Exceedence Probability P	(2) $K_{G_W,P}$	(3) a	(4) b	(5) $K^2_{G_W,P}-ab$	(6) K_P^U	(7) K_P^L	(8) $\bar{X}$	(9) S	(10) Q_P^U	(11) Q_P^L
0.5	0.04159	0.97059	−0.05583	0.23647	0.28648	−0.20079	3.3378	0.6528	3,348	1,609
.2	.85136	.97059	.66725	.27783	1.16340	.59091	3.3378	.6528	12,510	5,291
.1	1.25170	.97059	1.50919	.31929	1.61860	.96066	3.3378	.6528	24,798	9,223
.04	1.66164	.97059	2.70349	.37023	2.09343	1.33055	3.3378	.6528	50,628	16,083
.02	1.91729	.97059	3.61844	.40494	2.39260	1.55817	3.3378	.6528	79,376	22,645
.01	2.14117	.97059	4.52705	.43669	2.65597	1.75613	3.3378	.6528	117,928	30,493

$$b = K^2_{G_W, P} - \frac{Z_c^2}{N} \quad \text{(Note that } b \text{ varies as a function of annual probability } P\text{)}$$

The equation for computing points that define the lower confidence band level is:

$$K_P^L = \frac{K_{G_W, P} - (K^2_{G_W, P} - ab)^{0.5}}{a} \tag{12}$$

Continuing the example computations:

$$a = 1 - \left(\frac{Z_c^2}{2(N-1)}\right) = 1 - \left(\frac{(1.64485)^2}{2(47-1)}\right) = 0.97059$$

$$b_{P=0.5} = K^2{}_{G_W, P=0.5} = 0.04159^2 - 0.05756 = -0.05583$$
$$b_{P=0.2} = K^2{}_{G_W, P=0.2} = 0.085136^2 - 0.05756 = 0.66725$$
$$b_{P=0.1} = K^2{}_{G_W, P=0.1} = 1.25170^2 - 0.05756 = 1.50919$$
$$b_{P=0.04} = K^2{}_{G_W, P=0.04} = 1.66164^2 - 0.05756 = 2.70349$$
$$b_{P=0.02} = K^2{}_{G_W, P=0.02} = 1.91729^2 - 0.05756 = 3.61844$$
$$b_{P=0.01} = K^2{}_{G_W, P=0.01} = 2.14117^2 - 0.05756 = 4.52705$$

Table 7-3 shows the salient features of the 5- and 95-percent confidence band calculations.

It now remains to transpose the discharge relationship from the gauging station downstream to the Foss Dam site. This is accomplished by multiplying each discharge value by the ratio of the square roots of the drainage areas:

$$\frac{\sqrt{\text{Foss Dam Drainage Area}}}{\sqrt{\text{Gauged Drainage Area}}} = \frac{\sqrt{1{,}466 \text{ mi}^2}}{\sqrt{792 \text{ mi}^2}} = 1.3605$$

The final results representing the discharge-probability relationships are shown in table 7-4.

7.12 Example on Mixed Population Analyses

Consider the North Fork of the Big Thompson River near Drake, Colorado. The North Fork, lying on the eastern slope of the Rocky Mountains, heads at Rowe Glacier in Rocky Mountain National Park at an elevation of over 13,000 feet. The river descends to 6,170 feet at the gauging station near Drake. The record available for this analysis covers 30 years, from 1947 through 1976; and the drainage area involved is 82.2 square miles. The river is subject to flooding from two sources,

Table 7-4.—Discharge-probability relationships at Foss Dam[1].

P Annual Exceedance Probability	Q, ft³/s	Q_P^U, ft³/s	Q_P^L, ft³/s
0.5	3,152	4,555	2,189
.2	10,649	17,020	7,199
.1	19,436	33,726	12,548
.04	35,994	68,880	21,881
.02	52,857	107,993	30,809
.01	74,005	160,443	41,486

[1]Discharge values shown are to nearest cubic foot per second to facilitate reader's understanding. Normally, these values would be rounded to the nearest 10 ft³/s for values up to 10,000 ft³/s, and to the nearest 100 ft³/s for values over 10,000 ft³/s.

snowmelt and summer thunderstorm. A paper by Elliot, et al. [66] provides a listing of the annual rain peak and snow peak discharges, see table 7-5. Using these data, a separate discharge-probability analysis can be made for both the snowmelt and rainfall runoff related data. This is accomplished in the same manner shown in the example on Foss Dam, section 7.11, for the base frequency curve. The results of the analyses are shown in table 7-6. Using the parameters shown in table 7-6 in equation (8), section 7.11, leads to the individual discharge probability curves for rain floods and snowmelt floods shown on figure 7-4. Since it is desired to have a single curve that reflects the annual probability of a given level of flow occurring, the two curves must be combined. This is accomplished by applying equation (3), section 7.7. By applying this equation at a number of flow levels yields the points necessary to define the total population curve shown on figure 7-4.

Table 7-5.—Annual rain peak and snow peak discharges for North Fork of Big Thompson River near Drake, Colorado. From [66].

Water Year	Rain Peak		Snow Peak	
	Date	Discharge, ft^3/s	Date	Discharge, ft^3/s
1947	July 16	158	June 21	410
1948	Oct 15	86	11	166
1949	June 04	820	06	766
1950	July 10	450	12	129
1951	Aug 03	211	19	232
1952	June 26	159	05	283
1953	19	206	13	223
1954	July 15	44	27	47
1955	Aug 14	114	25	51
1956	May 21	228	02	161
1957	July 29	850	08	334
1958	June 25	147	May 24	295
1959	July 31	52	June 20	153
1960	Sep 18	43	18	120
1961	Aug 01	131	03	275
1962	July 12	138	15	110
1963	Aug 11	107	16	157
1964	July 29	74	08	84
1965	June 16	1,290	10	250
1966	July 20	584	18	60
1967	07	131	21	148
1968	Aug 12	156	06	148
1969	May 07	800	17	293
1970	July 22	194	24	251
1971	19	79	19	227
1972	June 17	134	04	170
1973	May 06	355	11	398
1974	July 15	156	17	184
1975	June 18	845	16	216
1976	July 31	8,710	09	82

Table 7-6.—Flood frequency parameters for North Fork of Big Thompson River near Drake, Colorado.

	Logarithmic Mean	Logarithmic Standard Deviation	Logarithmic Skew
Rainfloods:			
Generalized Skew			0.22
Systematic Record	2.3225	0.4877	1.3710
Weighted Skew			0.30
Final Parameters	2.3225	0.4877	0.30
Snowmelt Floods:			
Generalized Skew			−0.10
Systematic Record	2.2507	0.2738	−0.2522
Weighted Skew			−0.10
Final Parameters	2.2507	0.2738	−0.10

Chapter 8
FLOOD STUDY AND FIELD RECONNAISSANCE REPORTS

8.1 General

The importance of providing complete technical documentation of flood hydrology studies cannot be overemphasized. At any time after a flood study report is completed, any hydraulic engineer should be able to independently reproduce all flood values contained in the report based solely on its contents. If the documentation is complete, the reviewer will be in a better position to recommend approval of the study without having to resort to numerous contacts with the author to resolve points that are unclear. Each assumption and decision made in the course of the study should be thoroughly documented along with the rationale for making each assumption. The report should also include the field reconnaissance report as an appendix. All calculations leading to the development of a PMF hydrograph or discharge-probability relationship should either be included in the body of the report or in an appropriate appendix. In addition, all review comments and correspondence indicating approval of the study should be included as an appendix.

8.2 Specific Contents of a Flood Study Report

To achieve the objectives discussed in section 8.1, the following items should be included in all flood study reports.

(a) Authority.—The appropriate legislation, formal request, or contract for the report should be cited, including their dates. At the onset of a report, it is of considerable value to include a discussion on the objective of the study and a description of the physical features of the area involved. If the study is to support project planning efforts, a discussion on alternative physical project configurations should also be presented at this time. This will provide the reviewer the opportunity to assess whether the required hydrologic and meteorologic combinations have been properly recognized and analyzed. The level of the investigation or design that the study is conducted to support should also be presented.

(b) Summary of Results.—A summary of the study results should be presented near the beginning of the report to provide ready reference. In many cases, more than one PMF will be developed to reflect the seasonal variation of the magnitudes of the events. The peaks and volumes for each PMF derived are presented for the particular season (spring rain-on-snow, summer, fall, or winter rain-on-snow) and the type of storm that produced the flood hydrograph (snowmelt, general storm, local thunderstorm, etc.). In some flood studies, hydrographs of specific frequency or probability will have been developed, as discussed in chapter 7. If this is the case, the peaks and volumes for each associated probability should be tabulated. The final item in the summary should be a statement regarding the criteria to be used for routing the flood through the reservoir. These criteria should include the starting reservoir water surface elevation to be assumed at the onset of the PMF; release capability of outlet works and powerplant, if any, to be assumed during the routing; any constraints on spillway operations for gated or stoplogged structures; and any transbasin diversions that may be entering the reservoir.

*(c) **General Background.***—This section of the flood study report should begin with a brief description and purposes of previous flood studies prepared for locations at or near the site under investigation. A summary of the resulting flood values including peaks and volumes for PMF's and floods of specific frequency, if appropriate, should also be included for these previous studies. This should be followed with a discussion that includes all formal and informal agreements reached between the various organizational elements relative to technical aspects. Reference to the field reconnaissance should be made and should include the dates it was conducted, participants, and the offices the participants represented. The field reconnaissance report, the content of which will be discussed later in this chapter, should be included as an appendix to the flood hydrology report.

*(d) **Description of Drainage Basin.***—Much of the information that should be contained in this section may be extracted from the field reconnaissance report. The information should include a narrative description of the geographic location of the drainage basin, and should always include a location map. This may should be of sufficient scale so that readily identifiable geographic features such as highways, towns, and watercourses will enable the reader to easily identify the general location of the drainage basin. The drainage basin boundary should also be clearly delineated on the location map. The discussion should then proceed to fully describe the topographic features of the basin. This should include the range of elevations and stream slopes present in the basin, drainage network development and type, general geologic setting, prevalent soil types, and vegetal cover. The degree and type of development or land use, both present and projected, should always be included. Any projected development should reflect those of the most authoritative local source available, such as city or county planning and zoning entities. This section of the report should conclude by documenting the existing and future water control facilities in the basin. The potential effect of these facilities on flood runoff should be discussed, and the pertinent structural and hydraulic capacity and sizing data for each facility should be presented in tabular form.

*(e) **Historical Floods.***—This section of the flood study report should provide a discussion on the historical floods that have occurred in the basin under study and in nearby hydrologically and meteorologically similar basins. The discussion should include information relative to the peak discharge volumes, including their durations. Precipitation amounts and their distribution in time and space should also be included. The nature of the events should be described in terms of their rates of rise, fast or slow rising. Hydrographs, isohyetal maps, and mass curves of rainfall should be included, if available.

*(f) **Hydrometeorological Analysis.***—This analysis should begin with a discussion on the climatology of the region and specifically of the basin. Summary information relative to general air mass circulation, storm types typical of the region that impact on the drainage basin, and average and extreme temperature and precipitation experienced in the basin should be included; the latter should be tabulated by season. In virtually all cases, the PMP or storm will be derived by application of criteria contained in the appropriate hydrometeorological report discussed in chapter 3. The body of the flood hydrology report should present the pertinent data resulting from these applications. However, the work sheets and a brief narrative of the process involved in deriving the PMP estimate for each case should be included as an appendix to the report. There will be situations where the criteria contained in the HMR series reports do not apply, as discussed in chapter 3. This is generally encountered when performing hydrologic investigations for drainage basins larger in areal extent than considered in the report series. In these cases, the hydrometeorological analysis used in developing the individual drainage estimate should be included in its entirety as an appendix to the flood hydrology report. However, pertinent data from the individual drainage estimate should be included in the body of the report.

*(g) **Unit Hydrograph.***—If a synthetic unit hydrograph has been used in the flood study, a summary of the unit hydrograph lag parameters and the dimensionless hydrographs should be presented along with a brief narrative documenting the rationale for their selection. Since details of their selection are presented in the field reconnaissance report, which is included as an appendix to the flood study report, reference should be made to this report. In cases where the basis of the unit hydrograph used in the study is the result of an observed flood hydrograph reconstitution, the report should include all details of the reconstitution analysis. These details would include the estimated storm isohyetal pattern, temporal distribution of rainfall, infiltration loss rate assumed, base and interflow assumptions, and the final unit hydrograph used to produce the reconstituted flood hydrograph. A graphical representation of the reproduced flood hydrograph versus the observed flood hydrograph should always be presented. Also to be included on this graphical representation is a hyetograph of the basin average rainfall, with the loss

rates superimposed, and the component hydrographs (base flow, interflow, and surface runoff). An example of this type of presentation is shown on figure 4-14 in chapter 4.

(h) Infiltration Loss Rates.—This part of the flood study report basically reiterates, in summary fashion, the conclusions reached relative to infiltration loss rates determined to be appropriate in the course of the field reconnaissance or in the flood hydrograph reconstitution analyses. This summary should include a brief discussion on the soil types present in the drainage basin, general geologic setting, and consideration of vegetal cover as it impacts infiltration losses. The discussion should also include consideration of snowcover and frozen ground as they impact infiltration loss rates. In developed drainage basins, or where development is projected, and where the land use is predominantly urban, the summary should include information relating to the density of development and the resulting assumptions regarding composite loss rates reflecting the degree of pervious and impervious areas existing and projected.

(i) Base Flow and Interflow.—Included in this section should be the complete rationale for selection of the base flow and interflow hydrograph. Include an explanation on the recession flow typical for the season in which the PMF could occur, or an explanation of any substantial interflow that is likely to occur due to topographic, soils, or geologic conditions that may exist in many drainage basins. In some cases, the base flow component may consist of snowmelt runoff. If so, the rationale for determining the snowmelt runoff hydrograph, both peak and volume, and its temporal placement in relationship to the probable maximum rainflood hydrograph should be fully documented. In situations where the base flow and interflow have been concluded to be negligible, a full explanation of the rationale for reaching this conclusion should be documented.

(j) Snowmelt Runoff.—When the Western Snowmelt Equation (sec. 4.3, cp. 4) is used to estimate a drainage basin's snowmelt runoff potential, this part of the report should provide full documentation as to the depth, density, and areal extent of snow cover assumed, forest cover percentage determined to be representative of the basin, and wind velocities and temperatures assumed for the study. The report should then include the complete water budget analysis in tabular form, as described and illustrated in Engineering Monograph No. 35 [59][1]. In most cases, snowmelt runoff of a specific frequency or annual probability of occurrence will be adopted as the snowmelt component of the PMF rather than results obtained from the Western Snowmelt Equation. In this situation, a full explanation of the discharge probability study should be presented, and should include the streamflow records utilized, statistics

[1]Numbers in brackets refer to entries in the Bibliography.

generated based on the records, and the results of the analysis. In conclusion, the basis and rationale of the timing of the statistically determined snowmelt component with the probable maximum rainflood component should be provided.

(k) Antecedent Flood.—This section of the report should provide the basis and rationale for selecting the antecedent flood, particularly in reference to its magnitude and timing as related to the PMF. The hydrologic engineer should keep in mind that assumptions of reasonable meteorologic conditions are basic to antecedent flood estimates. Therefore, the report should include justification, in meteorological terms, for assumed conditions producing the antecedent flood. Generally, this justification should be based on consultation with hydrometeorologists in the Flood Section, Denver Office until such time as regionalized antecedent storm and flood criteria become available. The only criteria currently available for the Western States are found in NWS Technical Memorandum [61].

(l) Flood Hydrograph Routings.—As discussed in chapters 4 and 5, many flood hydrology studies are based on dividing basins into smaller subbasins that require channel routing and combining of the individual flood hydrographs. This section of the report should provide documentation on the type of flood routing technique applied, the basis and rationale for parameter selection, and results of the flood routing and combining performed. In some cases, a routing reconstitution of an observed flood event may have been conducted to provide site specific routing parameters. The results of such reconstitutions should be summarized in the report, and the details included as an appendix.

(m) Reservoir Routing Criteria.—The summary information regarding this item is included at the beginning of the report under "Summary of Study Results." This section of the report should include a detailed discussion on the basis and rationale for the starting reservoir water surface assumed to occur at the onset of the PMF. Include the consideration of the season when the PMF is likely to occur and the attendant possibility of the reservoir being at or above certain pool levels during this season. Transbasin diversions that may affect reservoir operations should also be discussed. Include any constraints that are to be assumed relative to the capability of the outlet works, power facilities, and spillways to pass or assist in accommodating the PMF. These constraints are generally determined by hydraulic design engineers knowledgeable in the area of hydraulic structures, and guidelines can generally be found in the Standard Operating Procedures for existing projects.

(n) PMF Hydrograph.—This section of the report provides a brief summary type discussion on the manner that each of the foregoing components are combined to yield the final flood hydrograph, or hydrographs in those cases where seasonal PMF's are determined, at the point of interest. To be included is the abstraction of infiltration losses from rainfall, unit hydrograph application to excess rainfall, addition of snowmelt, and base flow and interflow components. Conclude by summarizing, in tabular form, the PMF in terms of peak discharge and volumes over specified durations, usually 5-, 10-, and 15-day volumes. A graphical representation of the final flood hydrograph should be presented.

(o) Envelope Curves.—Each flood hydrology report should contain envelope curves of experienced peak discharge. Generally, relationships between drainage area, peak flood discharge, and volume amounts are presented. Each plot of data should be accompanied by tabular information as described in chapter 6. The narrative part of this section should provide a discussion on the stream gauges selected, representative geographic area, and the type (snowmelt, general storm, thunderstorm, etc.) of flood event represented.

(p) Discharge-Probability Analyses.—The flood hydrology study will usually include discharge-probability analyses. The results of these analyses are used for determining diversion requirements during construction of dams, cross drainage design for project roads and water conveyance facilities, and for flood control requirements, if required. This section should present the basis and results of the analysis leading to the development of peak and volume discharge-probability curves. The narrative portion should provide information relative to the source of the streamflow data used, length of record available at each of the stream gauges used, use of a regionalized approach (if applicable), and the type of flood event (general storm, snowmelt, etc.) represented by the curves. A summary table should be included that lists the peak discharge and appropriate volumes for the 5-, 10-, 25-, 50-, and 100-year events; separate tables should be provided for each type of flood event.

8.3 Field Reconnaissance Report

This report should be prepared by the Bureau regional or project office involved in the study as soon as practicable after the field reconnaissance is completed. This report should be based on the detailed field notes taken by each of the participants during the field reconnaissance. These notes should address each of the points discussed in chapter 4. When completed, the report should be transmitted to the Flood Section, Denver Office, for concurrence. In general, the field reconnaissance report should be included as an appendix to the flood hydrology report, and should specifically contain information regarding the following items:

- Dates of field reconnaissance, names of personnel on field reconnaissance team, and offices represented by these personnel.

- Locations and offices visited, individuals contacted, and routes traveled during reconnaissance.

• Applicable references to formal and informal correspondence that prompted the field reconnaissance.

• Synopsis of field trip, including a detailed discussion on observations made relative to defining the drainage basin's drainage network, topographic conditions, soil and geologic conditions, vegetal cover, land use, pertinent water control facilities, and any major obstructions to flow such as highway or railroad embankments that are located in the drainage basin. These observations should relate to the route traveled, and the discussion should be presented in sufficient detail so that it provides all necessary support for the conclusions reached relative to the on-site selection of hydrologic parameters.

• Conclusions as to the selection of hydrologic parameters including the unit hydrograph lag time coefficient used in the general lag equation, dimensionless unit hydrograph, infiltration rates and areas to which they are applicable, and the relative forest cover used for snowmelt analyses. In the event that members of the reconnaissance team cannot reach mutual agreement on the selection of these parameters, it should be so stated in the conclusions and referred to the Section Head, Flood Section, Denver Office, for resolution.

• A recommendation statement to the effect that the hydrologic parameters, determined to be appropriate, be adopted for use in the flood study.

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