PB85-182954

# HYDROLOGY

Research, Development, and Technology

Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101

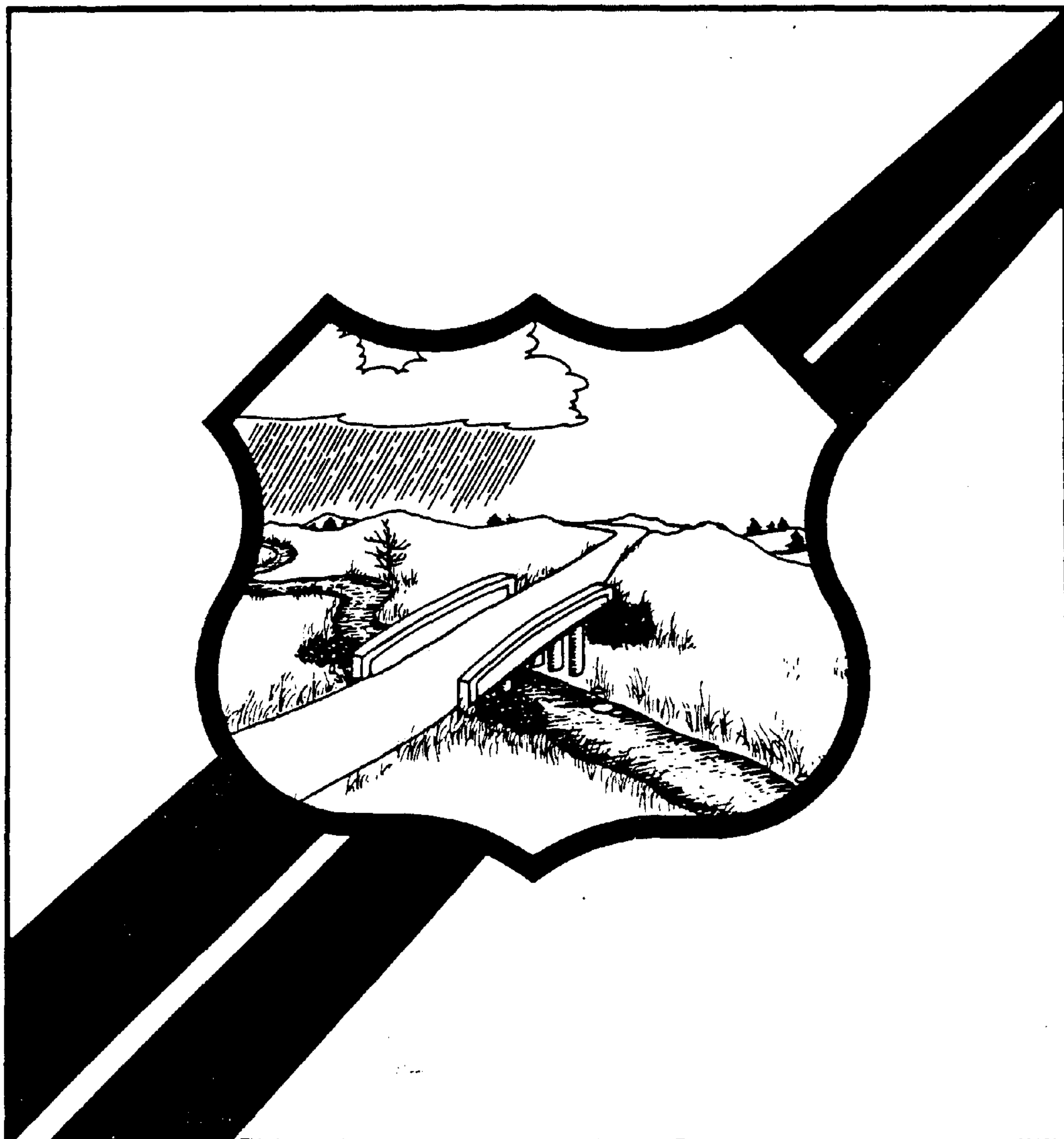
Hydraulic Engineering Circular No. 19

FHWA-IP-84-15

October 1984

U.S. Department of Transportation

**Federal Highway Administration**



FOREWORD

This Implementation Package provides practical hydrologic methods and techniques for the analysis and design of highway drainage structures. These procedures should be of interest to hydraulic, bridge, and highway design engineers.

The report was prepared by Stottler, Stagg, and Associates, Inc., with technical guidance from the Federal Highway Administration (FHWA) Office of Engineering Hydraulics Branch (HNG-31).

Sufficient copies of the report are being distributed to provide a minimum of one copy to each FHWA region office, division office, and each State highway agency. Additional copies will be available to public agencies from the FHWA Office of Engineering (HNG-31).

Donald C. Heim
for E. Dean Carlson
Director, Office of Engineering

R J Betsold
R. J. Betsold
Director, Office of Implementation

NOTICE

Technical Report Documentation Page

| | | |
|---|---|---|
| 1. Report No. HEC No. 19 FHWA-IP-84-15 | 2. Government Accession No. | 3. Recipient's Catalog No. PB85 182954/AS |
| 4. Title and Subtitle Hydrology | | 5. Report Date October 1984 |
| | | 6. Performing Organization Code |
| 7. Author(s) Frank D. Masch | | 8. Performing Organization Report No. |
| 9. Performing Organization Name and Address Stottler Stagg & Associates Architects, Engineers, Planners, Inc. 4545 Centerview, Suite 100 San Antonio, Texas 78228 | | 10. Work Unit No. (TRAIS) 35ZH028 |
| | | 11. Contract or Grant No. DTFH61-83-C-00118 |
| 12. Sponsoring Agency Name and Address Office of Implementation, HRT-10 Federal Highway Administration 6300 Georgetown Pike McLean, Virginia 22101 | | 13. Type of Report and Period Covered Design Manual Oct 83 - Oct 84 |
| | | 14. Sponsoring Agency Code |

15. Supplementary Notes

John M. Kurdziel (HRT-10), Contracting Officer's Technical Representative
Philip L. Thompson, Technical Assistance
Steven Chase, Technical Assistance

16. Abstract

This manual provides a synthesis of practical hydrologic methods and techniques to assist the highway engineer in the analysis and design of highway drainage structures. The manual begins with a discussion of descriptive hydrology, the surface runoff process and hydrologic data with emphasis given to the highway stream-crossing problem. The commonly used frequency distributions for estimating peak flows for basins with adequate data are discussed in detail and illustrated by examples. USGS regional regression equations and other methods for peak flow determinations in ungaged watersheds and in basins with insufficient data are presented with examples. Methods for developing unit hydrographs from streamflow data and by the Snyder and SCS synthetic procedures for ungaged sites are described in detail. Techniques for developing design storms and design hydrographs are given for basins with and without data. The Muskingum method for routing of hydrographs in channels and the Storage-Indication method for storage routing at highway embankments are discussed with illustrative examples. Estimates of peak flow and hydrograph development in urban watersheds using the SCS methods of TR-55 and the USGS Basin Development Factor procedure are illustrated in detail. The manual concludes with a brief discussion of risk analysis and its dependence on hydrologic analysis.

| | |
|---|---|
| 17. Key Words Hydrology, surface runoff, peak flows, frequency analysis, regression equations, hydrographs, unit hydrographs, design storms, routing, urban hydrology, risk analysis | 18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161 |

| 19. Security Classif. (of this report) | 20. Security Classif. (of this page) | 21. No. of Pages | 22. Price |
|---|---|---|---|
| Unclassified | Unclassified | 342 | |

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

## PREFACE

There is presently a very strong case for thorough hydrologic analysis by the highway engineer prior to project design. Such an analysis provides the necessary input for subsequent hydraulic design of drainage structures and information about the risks associated with discharges of given magnitudes. The resulting design is very much constrained by the information that the hydrologic analysis provides. It has been estimated that one-fifth of every highway construction dollar is expended on drainage related items. Clearly, in a program of highway design, construction, and operation which spends billions of dollars annually, any factor which appreciably affects drainage related costs is very important.

It is essential that highway drainage structures be economically designed. This means that the sizes of the drainage structures must be determined by a rational evaluation of all pertinent factors, such as initial capital costs, design life of the structures, the consequences of discharges of various magnitudes and durations, indirect costs and inconvenience to the traveling public and others. Such evaluations must be based upon the best estimate of discharges that the drainage structures will experience. This evaluation of discharges is the purpose of a hydrologic analysis and it is pivotal to economical drainage design.

The goals of this manual are two-fold. First, it presents the methods and techniques for estimating peak flows and hydrographs as used in traditional highway design. To this end, it includes many examples and illustrations of the required computational procedures. Secondly, it provides the highway designer with the capabilities to develop the hydrologic inputs for modern design methods utilizing risk analysis and least total expected cost techniques. In this respect, the manual is complementary to the Federal Highway Administration's HEC-17 "Design of Encroachments on Flood Plains Using Risk Analysis".

Hydraulic Engineering Circular No. 19 is divided into nine (9) sections, with references and appendices. The first section introduces the reader to the science of hydrology, the highway crossing design problem and various approaches to problem solution. Section 2 deals with the runoff process from precipitation through direct surface runoff and includes discussions of characteristics of rainfall events, hydrologic abstractions, effects of physical basin features, and characterization of runoff. Section 3 discusses sources of hydrologic data, data analysis, and adequacy of data. Statistical determinations of peak flow for basins with adequate data are treated in Section 4. The estimation of peak flows in basins with insufficient data and/or ungaged watersheds are discussed in Section 5. Hydrograph development is the subject of Section 6. Unit hydrographs are discussed together with the development of flood hydrographs from data and by synthetic methods for ungaged areas. The conversion of unit hydrographs to design hydrographs is explained. Section 7 discusses the routing of hydrographs with both channel and reservoir routing being covered. The effects of urbanization and other factors on peak flow hydrographs are included in Section 8. The USGS procedures and SCS TR-55 methods are thoroughly described. Section 9 presents a discussion of risk analysis as it applies to highway stream crossings. Each of the sections is illustrated and documented with appropriate examples.

This manual was prepared under contract DTFH61-83-C-00118 entitled "A Training Course Utilizing Micro-Computer Graphics on Hydrologic Design of Highway Stream Crossings". The author wishes to thank Mr. Vernon B. Sauer, Regional Surface Water Specialist, Southeastern Region, USGS, Atlanta, GA.; Dr. Stanley P. Sauer, Regional Hydrologist, Northeastern Region, USGS, Reston, VA; and Mr. Herman McGill, State Hydrologist, Soil Conservation Service, Temple, TX, who have provided reference material in support of this manual. Special thanks are also due to Mr. J. Dwight Reagan, Sr. Design Engineer, Texas Department of Highways and Public Transportation who has served as a Technical Advisor to the project and Mr. Bernie C. Massey, Supervisor Hydrologist, USGS, Texas District. These gentlemen have given their time extensively in the aquisition of data and review of this manuscript.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

## LIST OF FIGURES

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

## LIST OF TABLES

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

## Unit Conversion Factors

For those interested in using the metric system, the inch-pound units used in this manual may be converted to metric units by the following factors.

| From | | Multiply by | To obtain | |
|---|---|---|---|---|
| Unit | Abbrev. | | Unit | Abbrev. |
| cubic foot per second | CFS | 0.02832 | cubic meter per second | CMS |
| foot | ft | 0.3048 | meter | M |
| foot squared | $ft^2$ | 0.0929 | meter squared | $M^2$ |
| foot cubed | $ft^3$ | 0.0283 | meter cubed | $M^3$ |
| foot per mile | ft/mi | 0.189 | meter per kilometer | M/KM |
| inch | in | 2.54 | centimeter | CM |
| square mile | $mi^2$ | 2.59 | square kilo-meter | $KM^2$ |
| acre | | 0.4047 | hectare | |
| foot per second | FPS | 0.3048 | meter per second | MPS |

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

HYDROLOGY

# 1.0 INTRODUCTION

Hydrology is often defined as the science which deals with the physical properties, occurrence and movement of water in the atmosphere, on the surface of, and in the outer crust of the earth. This is an all-inclusive and somewhat controversial definition for there are individual bodies of science dedicated to study of various elements contained within this definition. Meteorology, oceanography, geohydrology, among others, are typical. For the highway designer, the primary focus is with the water that moves on the earth's surface and in particular that part which ultimately crosses transporation arterials, i.e. highway stream crossings.

Hydrologists have been studying the flow or runoff of water over land for many decades, and some rather sophisticated theories have been proposed to describe the process. Unfortunately, most of these attempts have been only partially successful not only because of the complexity of the process and the many interactive factors involved, but also because of the stochastic nature of rainfall, snowmelt and other sources of water. Most of the factors and parameters that influence surface runoff have been defined, but for many, complete functional descriptions of their individual effects exist only in empirical form. Extensive field data, empirically determined coefficients and sound judgment and experience are required for their quantitative analysis.

By application of the principles and methods of modern hydrology, it is possible to obtain solutions which are functionally acceptable and form the basis for the design of highway drainage structures. It is the purpose of this manual to present some of these principles and techniqes and to explain their uses by illustrative examples. First, however, it is desirable to discuss some of the basic hydrologic concepts that will be utilized throughout the manual and to discuss hydrologic analysis as it relates to the highway stream crossing problem.

## 1.1 Hydrologic Cycle

Water, which is found everywhere on the earth, is one of the most basic and commonly occurring substances. It is the only substance on earth that exists naturally in the three basic forms of matter, i.e. liquid, solid, and gas. The quantity of water varies from place to place and time to time. Although at any given moment the vast majority of the earth's water is found in the world's oceans, there is a constant interchange of water from the oceans to the atmosphere to the land and back to the ocean. This interchange is called the hydrologic cycle.

The hydrologic cycle, illustrated in Figure 1, is a description of the transformation of water from one phase to another and its motion from one location to some other. In this context, it represents the complete life cycle of water on and near the surface of the earth.

Beginning with atmospheric moisture, the hydrologic cycle can be described as follows. When warm moist air is lifted to the condensation level, precipitation in the form of rain, hail, sleet or snow falls on a watershed. Some of the water evaporates as it is falling and the rest either reaches the ground or is intercepted by buildings, trees and other vegetation. The intercepted water evaporates directly back to the atmosphere thus completing a part of the cycle. The remaining precipitation falls to the ground's surface or onto the water surfaces of rivers, lakes, ponds and the ocean.

If the precipitation falls as snow or ice, and the surface or air temperature is sufficiently cold, this frozen water will be stored temporarily as snowpack to be released later when the temperature increases and melting can occur. While contained in a snowpack, some of the water does escape through sublimation, the process where frozen water (i.e. ice) changes directly into water vapor and returns to the atmosphere without entering the liquid phase. When the temperature exceeds the melting point, the water from snowmelt becomes available to continue in the hydrologic cycle.

The water that reaches the earth's surface either evaporates, infiltrates into the root zone or runs off into puddles and depressions in the ground. The effect of infiltration is to increase the soil moisture. If the moisture content is less than the Field Capacity of the soil, water returns to the atmosphere through soil evaporation and by transpiration from plants and trees. If the moisture content becomes greater than the Field Capacity, the water percolates downward to become ground water. (Field Capacity is the moisture held by the soil after all excess gravitational drainage).

The part of precipitation which falls into puddles and depressions can evaporate, infiltrate, or if it fills the depressions, the excess water begins to flow overland until eventually it reaches natural drainageways. Water held within the depressions is called depression storage and is not available for overland flow or surface runoff.

Before flow can occur overland and in the natural and/or manmade drainage system, the flow path must be filled with water. This form of storage, called detention storage, is temporary since most of this water continues to runoff after the rainfall ceases. The precipitation that percolates down to ground water is maintained in the hydrologic cycle as seepage into streams and lakes, as capillary movement back into the root zone, or it is pumped from wells and discharged into irrigation systems, sewers or other drainageways. Water that reaches streams and rivers may be detained in storage reser-

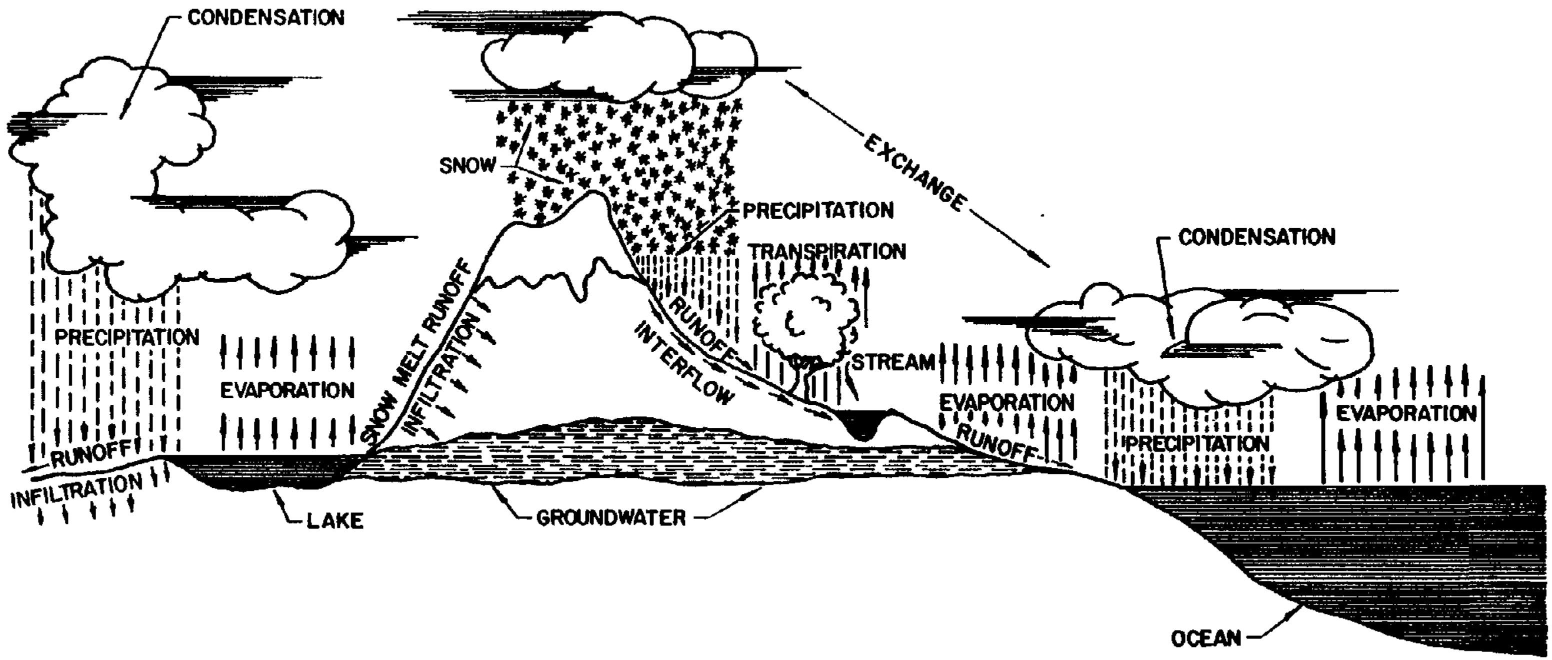



Figure 1. The Hydrologic Cycle

voirs and lakes or it eventually reaches the oceans. Throughout this path, water is continually evaporated back to the atmosphere, and the hydrologic cycle is repeated.

## 1.2 Hydrology of Highway Stream Crossings

In highway engineering, the diversity of drainage problems is broad and includes the design of bridges, culverts, siphons and other cross drainage structures for channels varying from small streams to large rivers. Stable open channels and stormwater collection and conveyance systems must be designed for both urban and rural areas. It is often necessary to evaluate the impacts of future land use, proposed flood control and water supply projects, and other planned and projected changes on the design of the highway crossing. On the other hand, the designer also has a responsibility to adequately assess flood potentials and environmental impacts that planned highway and stream crossings may have on the watershed.

### 1.2.1 Elements of the Hydrologic Cycle Pertinent to Highway Crossings

In highway design, the primary concern is with the surface runoff portion of the hydrologic cycle. Depending on local conditions other elements may be important, however, evaporation and transpiration can generally be discounted in highway design. The four most important parts of the hydrologic cycle to the highway designer are the following:

1. Precipitation
2. Infiltration
3. Storage
4. Surface Runoff

Precipitation is very important to the development of hydrographs and especially in synthetic methods and some peak discharge formulas where the flood flow is determined in part from excess rainfall or total precipitation less infiltration and storage. As described above, infiltration is that portion of the rainfall which enters the ground surface to become groundwater or to be used by plants and trees and transpired back to the atmosphere. Some infiltration may find its way back to the tributary system as interflow moving slowly near the ground surface or as groundwater seepage, but the amount is generally small. Storage is the water held on the surface of the ground in puddles and other irregularities (depression storage) and the water necessary to create a flow path (detention storage). Surface runoff is the water which flows across the surface of the ground into the watershed's tributary system and eventually into the primary watercourse.

The task of the designer is to determine the quantity and associated time distribution of runoff at a given highway stream crossing taking into account each of the pertinent aspects of the hydrologic cycle. In most cases, it is necessary to make reasonable approximations of these factors in the basic runoff determinations. In some situations, values can be assigned to storage and infiltration with confidence, while in others, there may be considerable uncertainty or the importance of one or both of these losses may be discounted in the final analysis. Thorough study of a given situation is necessary to permit assumptions to be made, and often only acquired experience or qualified advice permit solutions to the more complex and unique situations that may arise at a given crossing.

### 1.2.2 Basic Problems to the Hydrology of Highway Crossings

In any hydrologic analyses, there are normally three basic problems which include:

1. Measurement, recording, compilation and publication of data
2. Interpretation and analysis of data
3. Application to design or other practical problems.

The development of hydrology for a highway stream crossing is no different. Each of these problems must be addressed, at least in part, before an actual hydraulic structure can be designed. How extensively involved the designer becomes with each depends on the following:

1. Importance and cost of the structure or the acceptable risk of failure.
2. Amount of data available for the analysis.
3. Additional information and data needed.
4. Required accuracy.
5. Time and other resource constraints.

These factors normally determine the level of analysis justified for any particular design situation. As practicing designers will attest, they are often confronted with the problems of insufficient data and limited resources (time, manpower and money). It is impractical in routine design to use analytical methods that require extensive time and manpower or data not readily available or which are difficult to acquire. The more demanding methods and techniques should be reserved for those special projects where additional data collection and accuracy produces benefits which offset the additional costs involved. Examples of techniques requiring large amounts of time and data include basinwide computer simulation and rainfall-runoff models such as the Corps of Engineers' HEC-1, 1973, and the Soil Conservation Service's TR-20, 1965, among others. The discussion of such techniques is beyond the scope of the manual and the reader is referred to the List of References for more information on these models.

There are, however, a number of sound and proven methods available to analyze the hydrology for the more traditional and routine day to day design problem. These are procedures which enable peak flows and flow distributions (hydrographs) to be determined without an excessive expenditure of time and which use existing data, or in the absence of data, use synthetic methods to develop the design parameters. With care, and often with only limited additional data, these same procedures can be used to develop the hydrology for the more complex and/or costly design projects.

The choice of analytical method is a decision that must be made as each problem arises. For this to be an informed decision, the designer must know what level of analysis is justified, what data are available or must be collected, and what methods of analysis are available together with their relative strengths and weaknesses in terms of cost and accuracy.

Exclusive of the effects a given design may have upstream or downstream in a watershed, hydrologic analysis at a highway stream crossing requires the determination of either peak flow or the flood hydrograph, and in some cases both. Peak discharge (sometimes called the momentary maximum discharge) is critical because most highway stream crossing are traditionally designed to pass a given quantity of water with an acceptable level of risk. This capacity is usually specified in terms of the peak rate of flow during passage of a flood, called peak discharge or peak flow. Associated with this flow is a flood severity which is defined based on a predictable frequency of occurrence, i.e. a 10-year flood, a 50-year flood, etc. Table 1 is an example of some typical design frequencies for various hydraulic structures on certain classes of highways.

Table 1. Design Frequencies for Highway Structures

| Type of Structure | Design Frequency in Years: Interstate & Controlled Access Hyws. Main Lanes | Design Frequency in Years: Other Highways & Frontage Roads |
|---|---|---|
| Inlets and Sewers | 10 | 2 to 5 |
| Inlets for Depressed Roadways | 50 | 2 to 50 |
| Culverts | 50 | 2 to 10 |
| Small Bridges | 50 | 10 to 50 |
| River Crossings | 50 | 10 to 50 |

from Texas Highway Department, 1970.

Generally, the task of the highway designer is to determine the peak flows for a range of flood frequencies at a site in a drainage basin. Culverts, bridges or other structures are then sized to convey the design peak discharge within other constraints imposed on the design. If possible, the peak discharge which almost causes highway overtopping is estimated and this discharge is then used to evaluate the risk associated with the crossing.

Hydrograph development is important where a detailed description of the time variation of runoff is required. The concepts of risk analysis applied to design require that more than just peak flow be known. Similarly, the effects of urbanization, storage and other changes in a watershed affect flood flows in many ways. Travel time, time of concentration, runoff duration, peak flow and the volume of runoff may be changed by very significant amounts. The flood hydrograph is the primary way to evaluate and assess these changes. Additionally, when flows are combined and routed to another point along a stream, hydrographs are essential.

Neither peak flow or hydrographs present any real computational difficulties provided data are available for their determination. The common problem faced by the highway designer is that there may be insufficient flow data, or often no data at all, at the site for which a stream crossing is to be designed. While data describing the topography and the physical characteristics of the basin are readily attainable, rarely is there sufficient time to collect the flow data necessary to evaluate peak flows and hydrographs. In this case, the designer must resort to synthetic methods to develop design criteria. These methods require considerably more judgement and understanding in order to evaluate their application and reliability.

Finally, the designer must be constantly alert to changing or the potential for changing conditions in a watershed. This is especially important when reviewing reported streamflow data for a watershed which has undergone urban development, and channelization, diversions and other drainage improvements. Similarly, the construction of reservoirs, flow regulation measures, stock ponds and other storage facilities in the basin may be reflected in stream flow data. Other factors such as change in gage datum, moving of a gage, or mixed floods (floods caused by rainfall and snowmelt or rainfall and hurricanes) must be carefully analyzed to avoid misinterpretation and/or incorrect conclusions.

## 1.3 General Data Requirements

Regardless of the method selected for the analysis of a particular hydrologic problem, there is an almost immediate need for data. These needs take a variety of forms and may include data on precipitation and stream flow, information about the watershed, and the project to be designed. The type, amount and availability of the needed data will be determined in part, by the method selected for the analysis.

Section 3.0 of this manual deals extensively with hydrologic data. Types of data and information are discussed and the common sources for this information are identified. Other pertinent aspects on handling data are described including identification, documentation and indexing.

## 1.4 Solution Methods

Available analytical methods can be grouped into the two broad categories of deterministic and statistical methods. Deterministic methods strive to model the rainfall-runoff process while statistical methods utilize numerical data to describe the process. Deterministic methods can either be conceptual, where each of the elements of the runoff process is accounted for in some manner, or they may be empirical, where the relationship between rainfall and runoff is quantified based on measured data and experience. Statistical methods apply the techniques and procedures of modern statistical analysis to actual or synthetic data and define the needed design parameters directly.

### 1.4.1 Deterministic Methods

Deterministic methods often require a large amount of judgment and experience to be used effectively. These methods depend heavily upon the person applying the method and it is not uncommon for two different designers utilizing the same deterministic method to arrive at very different estimates of runoff for the same watershed. The accuracy of deterministic methods is also difficult to quantify. However, deterministic methods are usually based on fundamental concepts, and there is often an intuitive "rightness" about them which has led to their widespread acceptance in highway and other design practice. An experienced designer, familiar with a particular deterministic method, can arrive at reasonable solutions in a relatively short period of time.

### 1.4.2 Statistical Methods

Statistical methods, in general, do not require as much subjective judgment and experience to apply as deterministic methods. They are usually well documented mathematical procedures which are applied to measured or observed data. The answers a designer arrives at should be very nearly the same as those of another who applies the same procedures to the same data. The accuracy of statistical methods can also be measured quantitatively. However, statistical methods are not well understood, and as a result, answers are often misinterpreted. Another objective of this manual, and Section 4.0 in particular, is to present the commonly accepted statistical methods for peak flow determination in a logical format which encourages their use in highway drainage design.

# 2.0 RUNOFF PROCESS

From the discussion of the Hydrologic Cycle in Sec. 1.0, the runoff process can be defined as that collection of interrelated natural processes by which water, as precipitation, enters a watershed and then leaves as runoff. In other words, surface runoff is the excess precipitation which has not been removed from the watershed by any other process in the hydrologic cycle. The amount of precipitation which runs off from the watershed is defined as the "rainfall excess", and "hydrologic abstractions" is the commonly used term to group all the processes which extract water from the original precipitation. It follows then that surface runoff is equal to the rainfall excess, or in the case of the typical highway problem, the runoff is the original precipitation less infiltration and storage.

The primary purpose of this section is to describe more fully the runoff process. Pertinent aspects of precipitation are identified and each of the hydrologic abstractions is discussed in some detail. The important characteristics of runoff are then defined together with how they are influenced by different features of the drainage basin. The section concludes with a qualitative discussion of the runoff process beginning with precipitation and illustrating how this input is modified by each of the hydrologic abstractions.

## 2.1 Precipitation

Precipitation is the water which falls from the atmosphere in either liquid or solid form. It results from the condensation of moisture in the atmosphere due to cooling of a parcel of air. The most common cause of cooling is dynamic or adiabatic lifting of the air. Adiabatic lifting means that a given parcel of air is caused to rise with resultant cooling and possible condensation into very small cloud droplets. If these droplets coalesce and become of sufficient size to overcome the air resistance, precipitation in some form results.

### 2.1.1 Forms of Precipitation

Precipitation occurs in various forms. Rain is precipitation that is in the liquid state when it reaches the earth. Snow is frozen water in a crystalline state, while hail is frozen water in a 'massive' state. Sleet is melted snow which is an intermixture of rain and snow. Of course, precipitation that falls to earth in the frozen state cannot become part of the runoff process until thawing and melting occur. Much of the precipitation that falls in mountainous areas and in the northerly latitudes falls in frozen form and is stored as snowpack or ice until warmer temperatures prevail.

### 2.1.2 Types of Precipitation (by Origin)

Precipitation can be classified by the origin of the lifting motion which causes the precipitation. Each type is characterized by different spatial and temporal rainfall regimens. There are three major types of storms which can be classified as follows:

1) Convective Storms

2) Orographic Storms

3) Cyclonic Storms

A fourth type of storm is often added, the hurricane or tropical cyclone, although it is a special case of the cyclonic storm.

#### 2.1.2.1 Convective Storms

Precipitation from convective storms results as warm moist air rises from lower elevations into cooler overlying air as shown in Figure 2. The characteristic form of convective precipitation is the summer thunderstorm. The

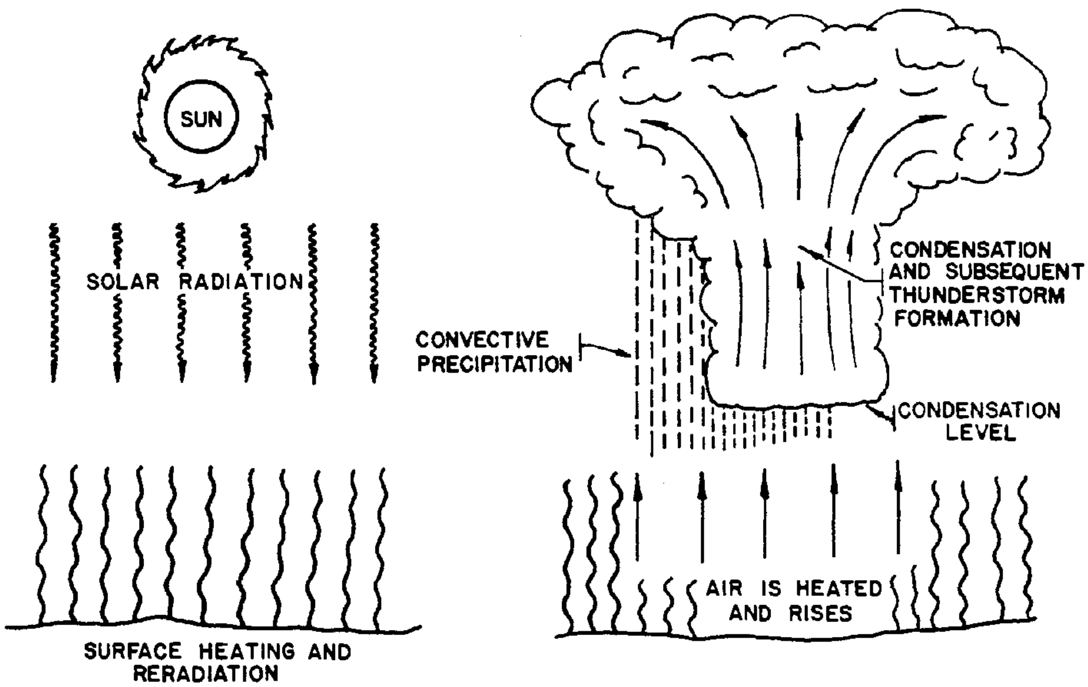



Figure 2. Convective Storm

surface of the earth is warmed considerably by mid-to late afternoon of a summer day, the surface imparting its heat to the adjacent air. The warmed air begins rising through the overlying air, and if proper moisture content conditions are met (condensation level), large quantities of moisture will be condensed from the rapidly rising, rapidly cooling air. The rapid condensation may often result in huge quantities of rain from a single thunderstorm spawned by convective action, and very large rainfall rates are quite common beneath slowly moving thunderstorms.

2.1.2.2 Orographic Storms

Orographic precipitation results as air is forced to rise over a fixed position geographic feature such as a range of mountains, Figure 3. The characteristic precipitation patterns of the Pacific coastal states are the result of significant orographic influences. Mountain slopes that face the wind (windward) are much wetter than the opposite (leeward) slopes. In the Cascade Range in Washington and Oregon, the west-facing slopes may receive upwards of 100 inches (254 cm) of precipitation annually, while the east facing slopes, only a few miles away over the crest of the mountains, receive on the order of 20 inches (51 cm) of precipitation annually.

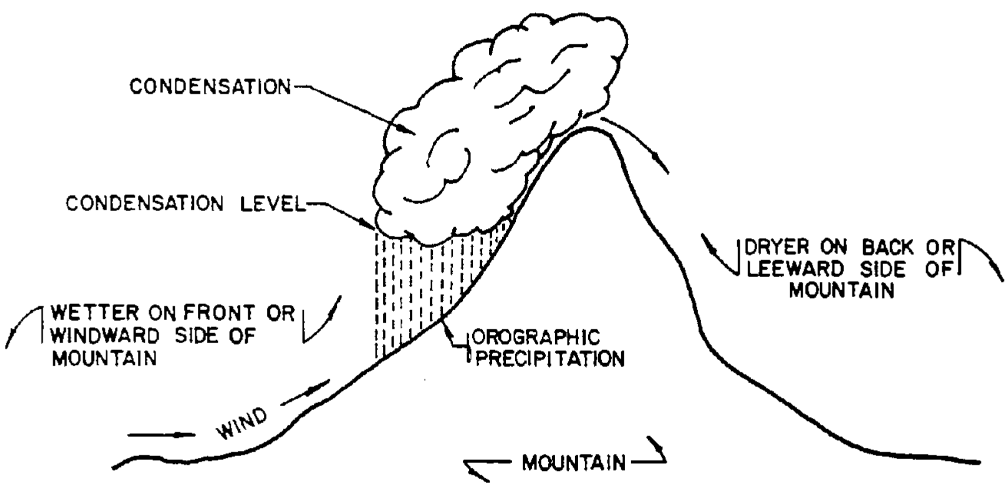



Figure 3. Orographic Storm

2.1.2.3 Cyclonic Storms

Cyclonic precipitation is caused by the rising or lifting of air as it converges on an area of low pressure. Air moves from areas of higher pressure toward areas of lower pressure. In the middle latitudes, cyclonic storms generally move from west to east and have both cold and warm air associated with them. These mid-latitude cyclones are sometimes called extra-tropical cyclones or continental storms. Continental storms occur at the boundaries of air of significantly different temperatures. A disturbance in the boundary between the two air parcels can grow, appearing as a wave as it travels from west to east along the boundary. Generally, on a weather map, the cyclonic storm will appear as shown in Figure 4 with two boundaries or fronts developed. One has warm air being pushed into an area of cool air, while the other has cool air pushed into an area of warmer air. This type of air movement is called a front; where warm air is the aggressor it is a warm front, and where cold air is the aggressor it is a cold front, Figure 5. The precipitation associated with a cold front is usually heavy and covers a relatively small area, whereas the precipitation associated with a warm front is more passive, smaller in quantity, but covers a much larger area. Tornadoes and other violent weather phenomena are associated with cold fronts.

#### 2.1.2.4 Hurricanes

Hurricanes or tropical cyclones develop over tropical oceans which have a surface water temperature greater than 85°F (29°C). A hurricane has no trailing fronts as the air is uniformly warm since the ocean surface from which it was spawned is uniformly warm. Hurricanes can drop tremendous amounts of moisture on an area in a relatively short time. Rainfall amounts of 15-20 inches (38-51 cm) in less than 24 hours are common in well-developed hurricanes, where winds are sustained in excess of 75 miles per hour (121 km/hr).

### 2.1.3 Characteristics of Rainfall Events

The characteristics of precipitation which are important to highway drainage are:

1. Intensity (rate of rainfall)

2. Duration

3. Time Distribution of Rainfall

4. Storm shape, size, and movement

5. Frequency

Intensity is defined as the rate of rainfall and is commonly given in the units of inches per hour. All precipitation is measured as the vertical depth of water (or water equivalent in the case of snow) which would accumulate on a flat level surface if all the precipitation remained where it had fallen. A variety of rain gages have been devised to measure precipitation. All first-order weather stations utilize gages that provide nearly continuous records of accumulated rainfall with time. These data are typically reported in either tabular form or as mass rainfall curves, Figure 6.

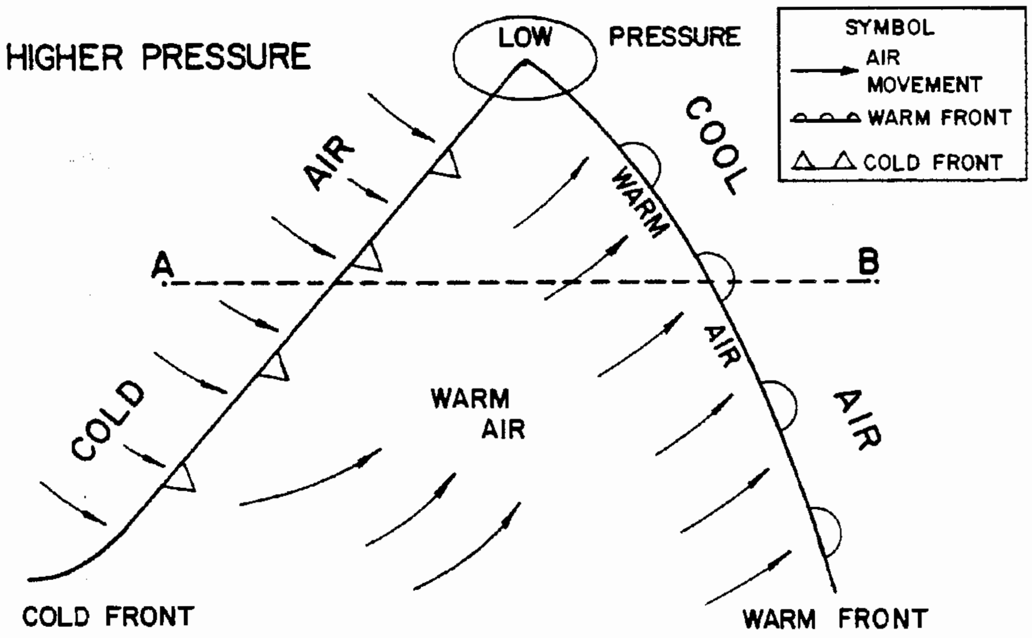



Figure 4. Storm As It Appears on Weather Map In The Northern Hemisphere

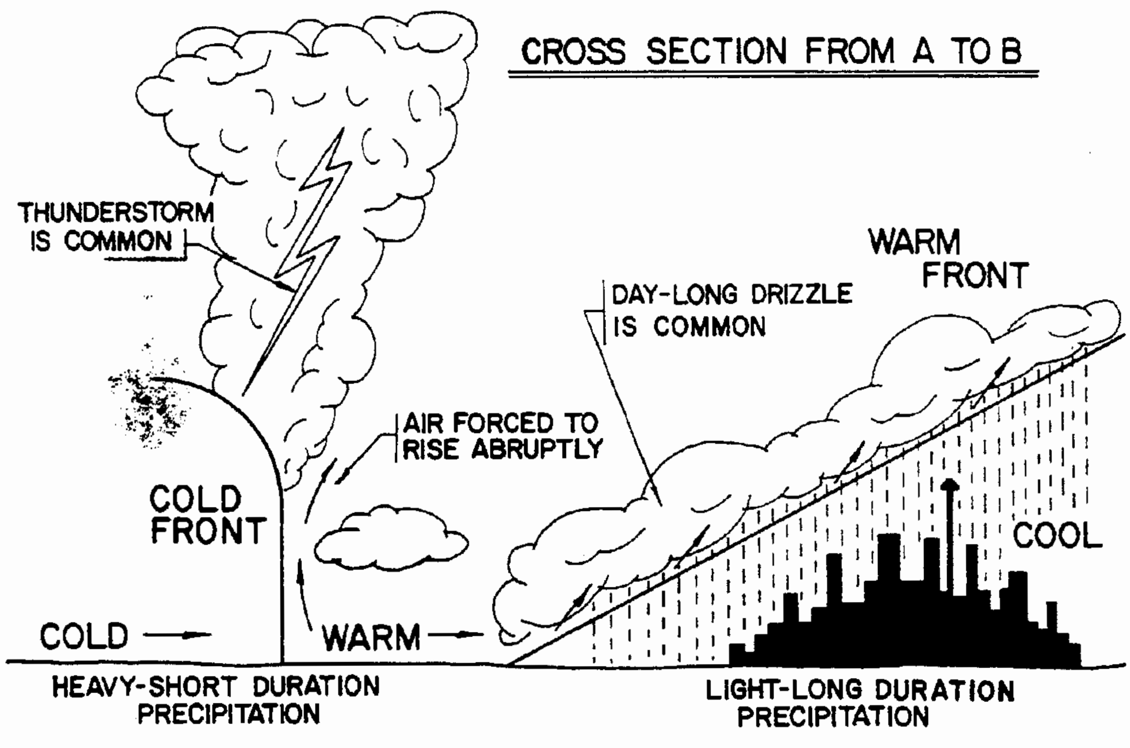



Figure 5. Cyclonic Storms In Mid-Latitude

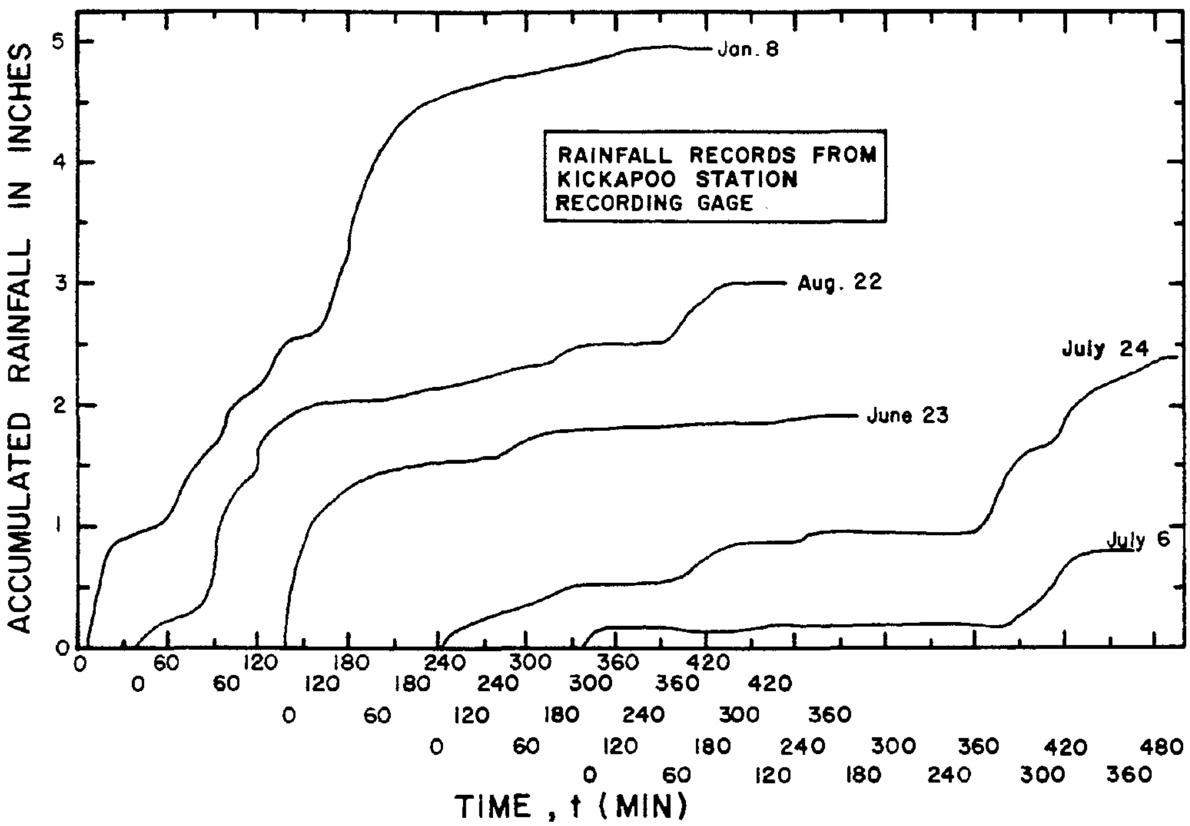



Figure 6. Mass Rainfall Curves

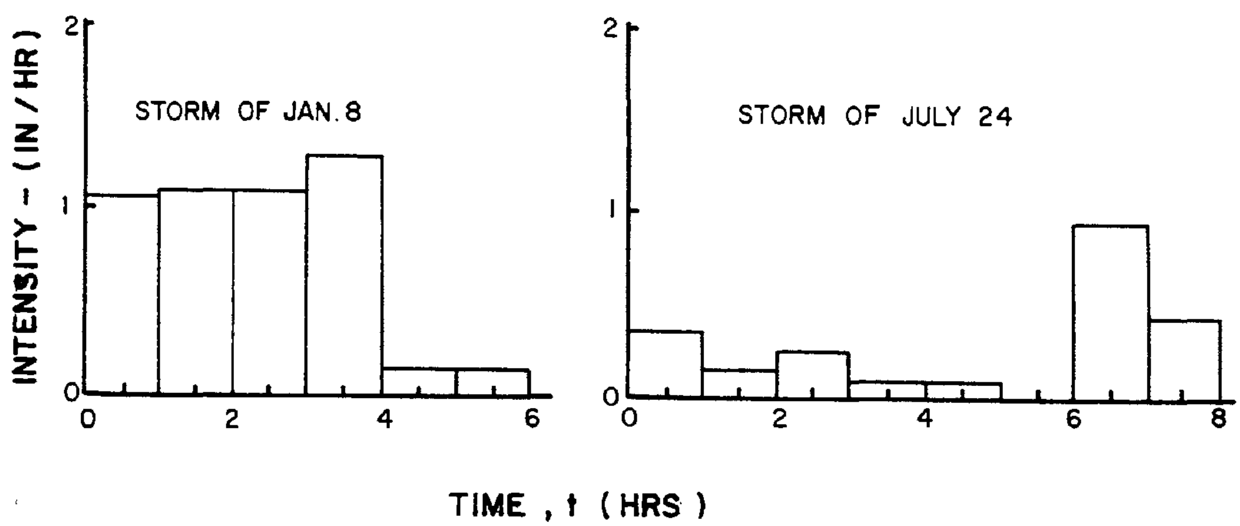



Figure 7. Rainfall Hyetographs for Kickapoo Station

In any given storm, the instantaneous intensity is the slope of the mass rainfall curve at a particular time. For hydrologic analysis, it is desirable to divide the storm into convenient time increments and to determine the average intensity over each of the selected periods. These results are then plotted as rainfall hyetographs, two examples of which are shown for Kickapoo Station in Figure 7.

While the above illustrations use a 1-hour increment to determine the average intensity, any time increment compatible with the time scale of the hydrologic event to be analyzed can be used. Figure 7 shows the irregular and complex nature of different storms even though measured at the same station.

In spite of this complexity, intensity is the most important of the rainfall characteristics. All other factors being equal, the more intense the rainfall, the larger will be the discharge from a given watershed. Intensities can vary from misting conditions where a trace (<0.005 inches total, or approximately .01 cm) of precipitation may fall to cloudbursts where several inches per hour are common. Figure 8, taken from the U.S. Weather Bureau, 1947, summarizes some of the maximum observed rainfalls in the United States.

The events given in Figure 8 are depth-duration values at a point and can only be interpreted for average intensities over the reported durations. Still some of these storms were very intense with average intensities on the order of 5 to 20 inches per hour (13 to 51 cm/hr) for the shorter durations (<1 hour) and from 2 to 10 inches per hour (5 to 25 cm/hr) for the longer durations (>1 hour). Since these are only averages, it is probable that intensities in excess of these values occurred during the various storms.

The storm duration or time of rainfall can be determined from either Figure 6 or 7. In the case of Figure 6, the duration is the time from the beginning of rainfall to the point where the mass curve becomes horizontal indicating no further accumulation of precipitation. In Figure 7, the storm duration is simply the width (time base) of the hyetograph. The most direct effect of storm duration is on the volume of surface runoff with longer storms producing more runoff than shorter duration storms of the same intensity.

The time distribution of the rainfall is normally given in the form of intensity hyetographs similar to those shown in Figure 7. This time variation directly determines the corresponding distribution of the surface runoff. As illustrated in Figure 9, high intensity rainfall at the beginning of a storm, such as the January 8 storm in Figure 7, will result in a rapid rise in the runoff followed by a long recession of the flow. Conversely, if the more intense rainfall occurs toward the end of the duration, as in the July 24 storm of Figure 7, the time to peak will be longer followed by a rapidly falling recession.

Storm shape, size and movement are normally determined by the type of storm, Sec. 2.1.2. For example, storms associated with cold fronts (thunderstorms) tend to be more localized, faster moving and of shorter duration, whereas warm fronts tend to produce slowly moving storms of broad areal extent and

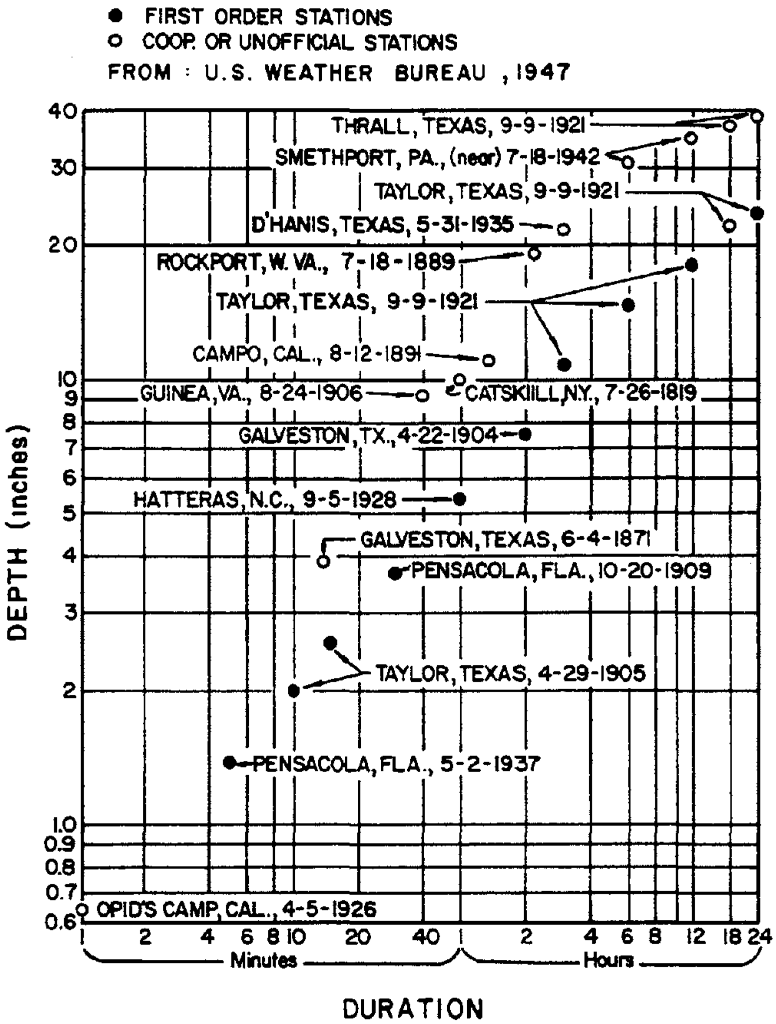



Figure 8. Maximum Observed U.S. Rainfalls

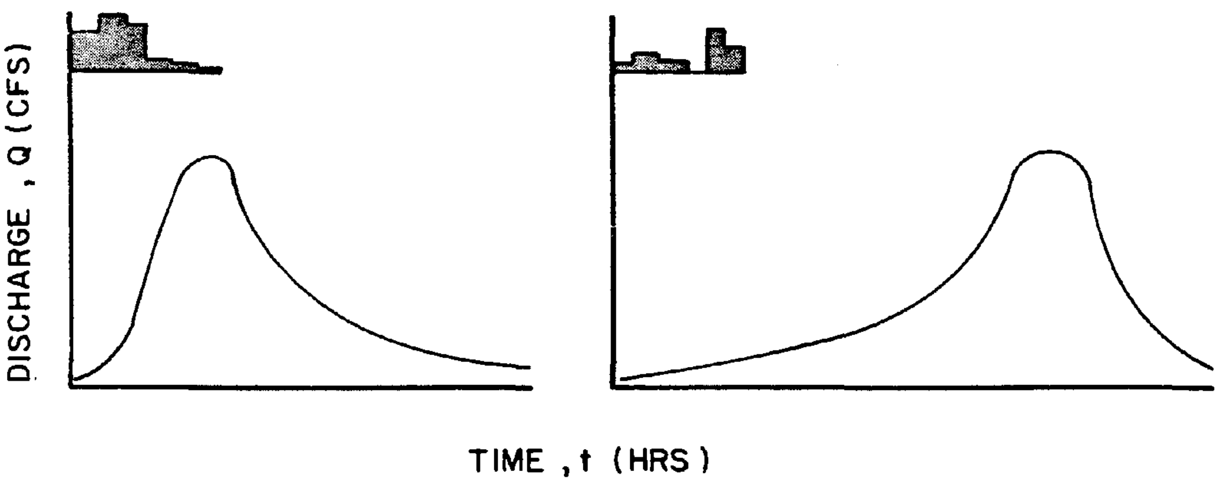



Figure 9. Effect of Time Variation of Rainfall Intensity on the Surface Runoff

longer durations. All three of these factors determine the areal extent of precipitation and how large a portion of the drainage area contributes over time to the surface runoff. As illustrated in Figure 10, a small localized storm of a given intensity and duration, over a part of the drainage area will result in much less flow than if the same storm covered the entire watershed. The location of a localized storm in the drainage basin also affects the time distribution of the surface runoff. A storm near the outlet of the watershed will result in the peak flow occurring very quickly and a rapid passage of the flood. If the same storm occured in a remote part of the basin, the runoff at the outlet would be longer and the peak flow lower due to storage in the channel.

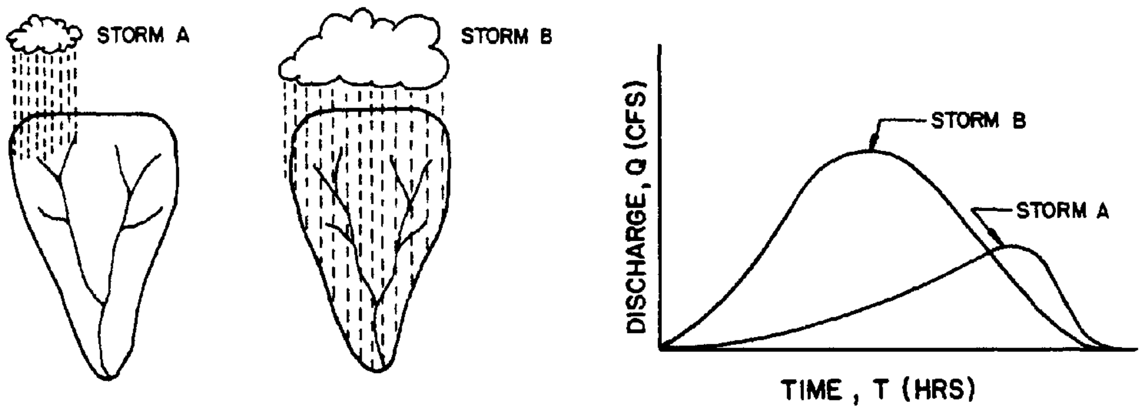



Figure 10. Effect of Storm Size on Surface Runoff

Storm movement has a similar effect on the runoff distribution particularly if the basin is long and narrow. Figure 11 shows that a storm moving up a basin from its outlet gives a distribution of runoff that is relatively symmetrical with respect to the peak flow. The same storm moving down the basin will usually result in a higher peak flow and an unsymmetrical distribution with the peak flow occurring later in time.

Frequency is also an important characteristic because it establishes the frame of reference for how often precipitation with given characteristics is likely to occur. From the standpoint of highway design, a primary concern is with the frequency of occurrence of the resulting surface runoff, and in particular, the frequency of the peak discharge. While the designer is cautioned about assuming that a given frequency storm always produces a flood of the same frequency, there are a number of analytical techniques that are based on this assumption, particularly for ungaged watersheds. Some of the

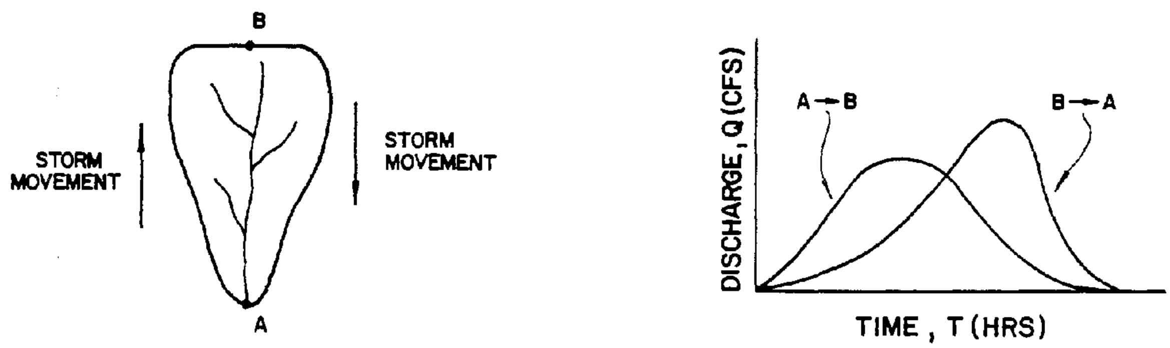



Figure 11. Effect of Storm Movement on Surface Runoff

factors that determine how closely the frequencies of precipitation and peak discharge correlate with one another are discussed in Sec. 2.4.

Precipitation is not easily characterized although there have been many attempts to do so. There are references and data sources available which provide general information on the character of precipitation at specified geographic locations. These sources are discussed more fully in Section 3.0 and Appendix C. It is important, however, to understand the highly variable and erratic nature of precipitation. Highway designers should become familiar with the different types of storms and the characteristics of precipitation which are indigenous to their regions of concern. They should also understand the seasonal variations which are prevalent in many areas. In addition, it is very beneficial to study reports which have been prepared on historic storms in a region. Such reports can provide information on past storms and the consequences they may have had on drainage structures.

## 2.2 Hydrologic Abstractions

Abstractions is the collective term given to the various processes which act to remove water from the incoming precipitation before it leaves the watershed as runoff. These processes are evaporation, transpiration, interception, infiltration, depression storage and detention storage.

### 2.2.1 Evaporation

Evaporation occurs continually whenever the air is unsaturated and temperatures are sufficiently high. Air is 'saturated' when it holds its maximum capacity of moisture at the given temperature. Saturated air has a relative humidity of 100 percent. Evaporation plays a major role in determining the long term water balance in a watershed. However, evaporation is usually insignificant in small watersheds for single storm events and can be discounted when calculating the discharge from a given rainfall event.

### 2.2.2 Transpiration

Transpiration is the physical removal of water from the watershed by the life actions associated with the growth of vegetation. In the process of respiration, green plants consume water from the ground and transpire water vapor to the air through their foliage. As was the case with evaporation, this abstraction is only significant when taken over a long period of time, and has minimal effect upon the runoff resulting from a single storm event for a small watershed.

### 2.2.3 Interception

Interception is the removal of water which wets and adheres to objects above ground such as buildings, trees and vegetation. This water is subsequently removed from the surface through evaporation. Interception can be as high as 0.06 inches (0.15 cm) during a single rainfall event but usually is nearer 0.02 inches (0.05 cm). The quantity of water removed through interception is usually not significant for an isolated storm but when added over a period of time, can be a significant. It is thought that as much as 25 percent of the total annual precipitation for certain heavily forested areas of the Pacific Northwest of the United States is lost through interception during the course of a year.

### 2.2.4 Infiltration

The most important abstractions in determining the surface runoff from a given precipitation event are infiltration, depression storage and detention storage. Infiltration is the flow of water into the ground by percolation through the earth's surface. The process of infiltration is complex and depends upon many factors such as soil type, vegetal cover, antecedent moisture conditions or the amount of time elapsed since the last precipitation event, precipitation intensity, and temperature. Infiltration is usually the single most important abstraction in determining the response of a watershed to a given rainfall event. As important as it is, there is no generally acceptable model developed to accurately predict infiltration rates for a given watershed.

### 2.2.5 Depression Storage

Depression storage is the term applied to water which is lost because it becomes trapped in the numerous small depressions which are characteristic of any natural surface. When ponded water accumulates in a low point with no possibility for escape as runoff, the accumulation is referred to as depression storage. The amount of water which is lost due to depression storage varies greatly with the land use. A paved surface will not detain as much water as a recently furrowed field. The relative importance of depression storage in determining the runoff from a given storm depends on the amount and intensity of precipitation in the storm. Typical values for depression storage range from 0.02 to 0.30 inches (0.05 to 0.8 cm) with some values as high as 0.50 inches (1.3 cm) per event.

### 2.2.6 Detention Storage

Detention storage is water which is temporarily stored in the depth of water necessary for overland flow to occur. In other words, the volume of water in motion over the land constitutes the detention storage. The amount of water which will be stored is dependent on a number of factors such as land use, vegetal cover, slope and rainfall intensity. Typical values for detention storage range from 0.1 to 0.4 inches (0.25-1.0 cm) but values as high as 2.0 inches (5.1 cm) have been reported.

It is evident that the runoff, if any, which results from a given precipitation event over a specific watershed is highly influenced by the abstractions. In order for the highway designer to understand the hydrology of a region, it is important to know the relative effect each of the abstractions identified above has on the response of typical watersheds to different types of storms.

## 2.3 Characteristics of Runoff

Water which has not been abstracted from the incoming precipitation leaves the watershed as surface runoff. While runoff occurs in several stages, the flow which becomes channelized is the main consideration to highway stream crossing design since it determines the size of a given drainage structure. The rate of flow or runoff at a given instant, in terms of volume per unit of time, is called discharge. Some important characteristics of runoff important to drainage design are:

1. peak discharge or peak rate of flow
2. discharge variation with time (hydrograph)
3. stage-discharge relationship
4. total volume of runoff
5. frequency with which discharges of specified magnitudes are likely to occur (probability of occurrence)

### 2.3.1 Peak Discharge

The peak discharge, often called peak flow, is the maximum flow of water passing a given point during or after a rainfall event. Highway designers are interested in peak flows for storms in an area because it is the discharge which a given structure must be sized to handle. Of course, the peak flow varies for each different storm, and it becomes the designer's responsibility to size a given structure for the magnitude of storm which is determined to present an acceptable risk in a given situation. Peak flow rates can be affected by many factors in a watershed, including rainfall, basin size and its physiographic features.

### 2.3.2 Time Variation (Hydrograph)

The flow in a stream varies from time to time, particularly during and in response to storm events. As precipitation falls and moves through the watershed, water levels in streams rise and may continue to do so (depending on position in the watershed) after the precipitation has ceased. The response of an affected stream through time during a storm event is characterized by the flood hydrograph. This response can be pictured by graphing the flow in a stream relative to time. The primary features of a typical hydrograph are illustrated in Figure 12 and include the rising and falling limbs, the peak flow, the time to peak and the time of flood. There are several types of hydrographs such as flow per unit area and stage hydrographs, but all display the same typical variation through time.

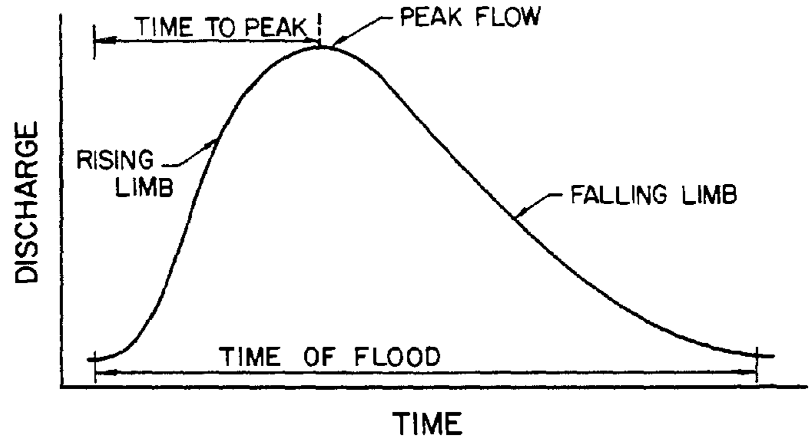



Figure 12. Flood Hydrograph

### 2.3.3 Stage-Discharge

The stage of a river is the elevation of the water surface above some arbitrary zero datum. The datum can be mean sea level, but usually is set slightly below the point of zero flow in the given stream. Discharge which is the quantity of water passing a given point is directly related to the stage of a river, Figure 13. As the stage rises the discharge increases, and conversely, as the stage falls the discharge decreases. Generally, discharge is related to stage at a particular point by a series of field measurements of discharge which define the stage-discharge relationship. The discharge is determined by mapping a cross-sectional area in a stream, and multiplying the area by point measurements of velocity at various locations and depths in that cross section. The average velocity in a given cross section segment (of not more than 10 percent of the total cross-sectional area of a stream) can be approximated within 2 percent by averaging the velocities at two-tenths and eight-tenths of the total depth at the measurement location. The velocity at six-tenths depth below the surface also characterizes the mean velocity in a cross-sectional segment within about 5 percent. The total discharge is the sum of the incremental flows estimated for each cross-sectional segment.

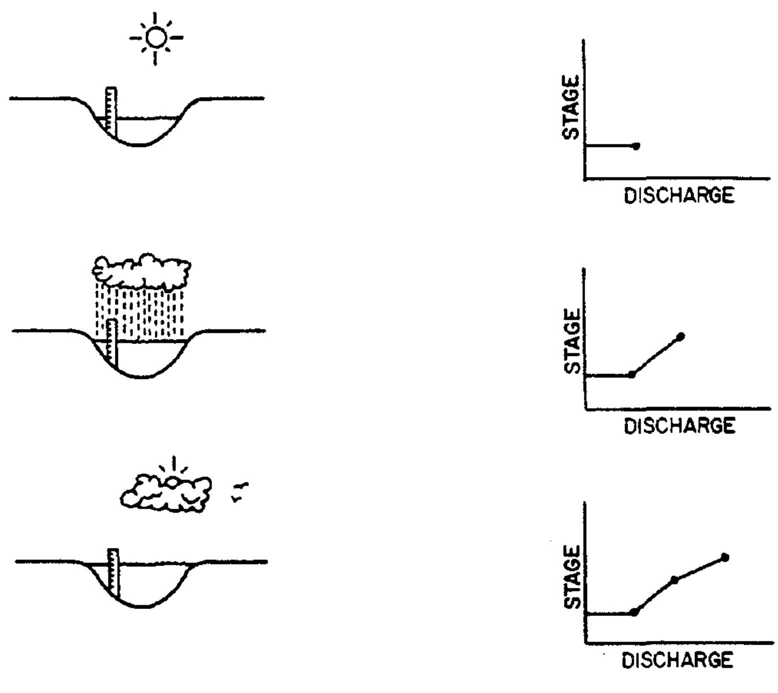



Figure 13. Relation Between Stage and Discharge

2.3.4 Total Volume

The total volume of runoff from a given flood is of primary importance to the design of storage facilities and flood control works. Flood volume is not normally a consideration in the design of highway structures although it is used in various analyses for other design parameters. Flood volume is most easily determined as the area under the flood hydrograph, Figure 12, and is commonly measured in units of cubic feet or acre-feet.

### 2.3.5 Frequency

Frequency is the number of times a flood of a given magnitude can be expected to occur on an average over a long period of time. By its definition, frequency is a probabilistic concept and is actually the probability that a flood of a given magnitude may be exceeded in a specified period of time, usually 1 year. Frequency is an important design parameter in that it identifies the level of risk acceptable for the design of a highway structure.

## 2.4 Effects of Basin Characteristics on Runoff

The spatial and temporal variations of precipitation and the concurrent variations of the individual abstraction processes determine the characteristics of the runoff from a given storm. These are not the only factors involved, however. Once the local abstractions have been satisfied for a small area of the watershed, water begins to flow overland and eventually into a natural drainage channel such as a gulley or a stream valley. At this point, the hydraulics of the natural drainage channels have a large influence on the character of the total runoff from the watershed.

There are many factors which determine the hydraulic character of the natural drainage system. A few of these are drainage area, slope, hydraulic roughness, natural and channel storage, stream length, channel density, antecedent moisture conditions, and other factors such as vegetation, channel modifications, etc. The effect that each of these factors has on the important characteristics of runoff is often difficult to quantify. The following paragraphs discuss some of the factors which affect the hydraulic character of a given drainage system.

### 2.4.1 Drainage Area

Drainage area is the most important watershed characteristic affecting runoff. The larger the contributing drainage area, the larger will be the flood runoff. Regardless of the method utilized to evaluate flood flows, drainage area is directly related to the peak flood flow.

### 2.4.2 Slope

Steep slopes tend to result in rapid responses to local rainfall excess and consequently higher peak discharges, Figure 14a. The runoff is quickly removed from the watershed, so the hydrograph is short with a high peak. The stage-discharge relationship is highly dependent upon the local characteristics of the cross section of the drainage channel, and if the slope is sufficiently steep, supercritical flow may prevail. The total volume of runoff is also affected by slope. If the slope is very flat, the rainfall excess will not be removed as rapidly. The process of infiltration will have more time to affect the rainfall excess, thereby resulting in a reduction of total volume.

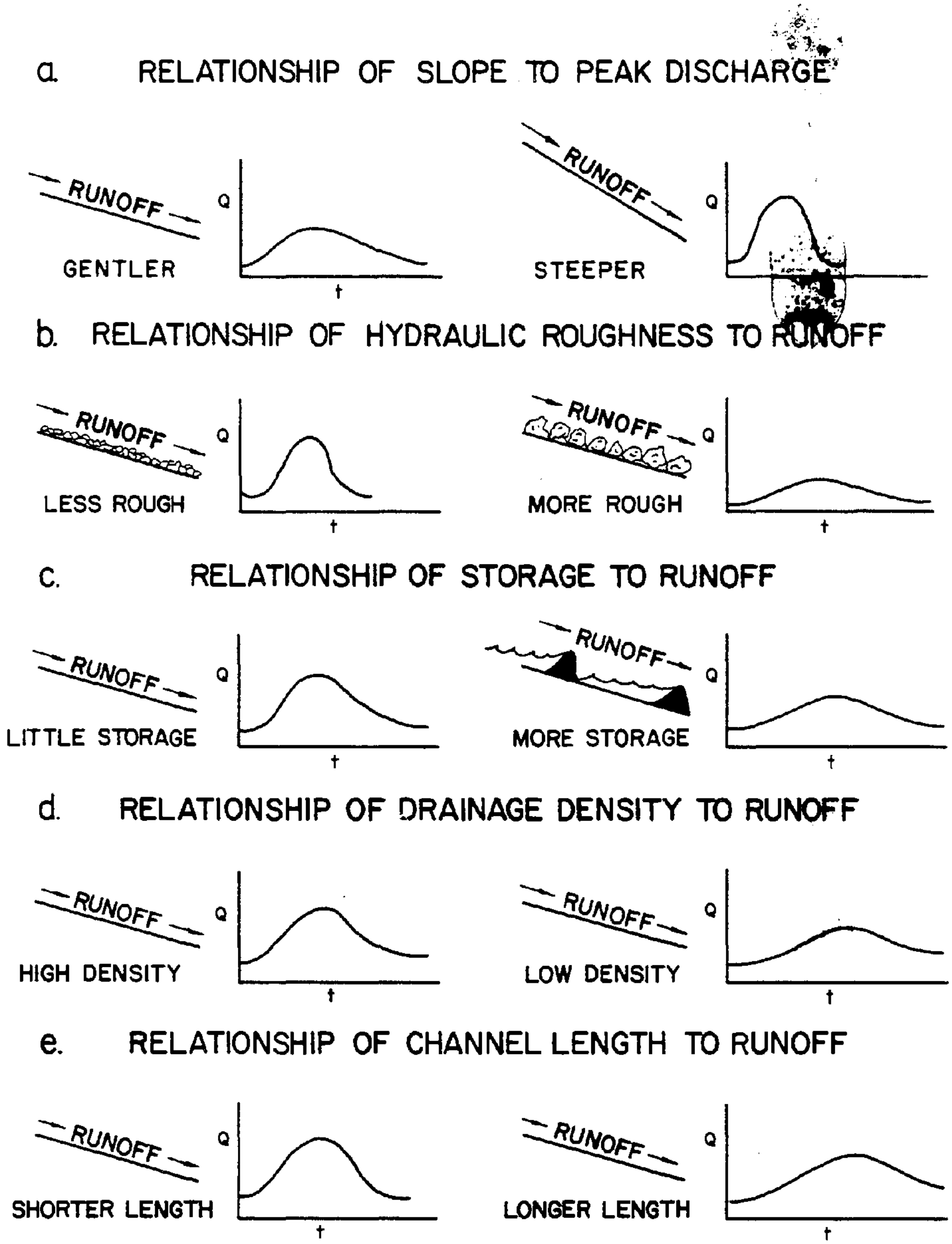



Figure 14. Effects of Basin Characteristics on the Flood Hydrograph

The effect of slope on the frequency of a discharge of given magnitude is not immediately obvious. Slope is very important in how quickly a drainage channel will convey water, and therefore it determines the sensitivity of a watershed to precipitation events of various time durations. Watersheds with steep slopes will rapidly convey incoming rainfall, and if the rainfall is convective (characterized by high intensity and relatively short duration), the watershed will respond very quickly with peak flow occuring shortly after the onset of precipitation. If these convective storms occur with a given frequency, then the resulting runoff can be expected to occur with a similar frequency. On the other hand, for a watershed with a flat slope, the response to the same storm will not be as rapid, and depending on a number of other factors, the frequency of the resulting discharge may be dissimilar to the storm frequency.

### 2.4.3 Hydraulic Roughness

Hydraulic roughness is a composite of the physical characteristics which influence the flow of water across the surface, whether natural or channelized. It affects both the time response of a drainage channel and the channel storage characteristics. Hydraulic roughness has a marked effect on the characteristics of the runoff resulting from a given storm. The peak rate of discharge is inversely proportional to hydraulic roughness, i.e., the lower the roughness, the higher the peak discharge. Roughness affects the runoff hydrograph in a manner opposite of slope. The lower the roughness, the more peaked and shorter in time the resulting hydrograph will be for a given storm, Figure 14b.

The stage-discharge relationship for a given section of drainage channel is also dependent on roughness (assuming normal flow conditions and the absence of artificial controls). The higher the roughness, the higher the stage for a given discharge.

The total volume of runoff is virtually independent of hydraulic roughness. An indirect relationship does exist in that higher roughnesses slow the watershed response and allow some of the abstraction processes more time to affect the runoff. Roughness also has an influence on the frequency of discharges of certain magnitudes by affecting the response time of the watershed to precipitation events of specified frequencies.

### 2.4.4 Storage

It is common for a watershed to have natural or man-made storage which greatly affects the response to a given precipitation event. Common features which contribute to storage within a watershed are lakes, marshes, heavily vegetated overbank areas, natural or manmade constrictions in the drainage channel which cause backwater, and the storage in the floodplains of large, wide rivers. Storage can have a significant effect in reducing the peak rate of discharge, although this reduction is not necessarily universal. There have been some instances where artificial storage redistributes the discharges very radically resulting in higher peak discharges than would have occurred had the storage not been added. As shown in Figure 14c, storage generally spreads the hydrograph out in time, delays the time to peak and alters the shape of the resulting hydrograph from a given storm.

The stage-discharge relationship also can be influenced by storage within a watershed. If the section of a drainage channel is upstream of the storage and within the zone of backwater, the stage for a given discharge will be higher than if the storage were not present. If the section is downstream of the storage, the stage-discharge relationship may or may not be affected, depending upon the presence of channel controls.

The total volume of water is not directly influenced by the presence of storage. Storage will redistribute the volume over time, but will not directly change the volume. By redistributing the runoff over time, storage may allow other abstraction processes to affect the runoff as was the case with slope and roughness.

Storage has a very definite effect upon the frequency of discharges of given magnitudes. It tends to dampen the response of a watershed to very short events and to accentuate the response to very long events. This alters the relationship between frequency of precipitation and the frequency of the resultant runoff.

### 2.4.5 Drainage Density

Drainage density can be defined as the ratio between the number of well defined drainage channels and the total drainage area in a given watershed. It is determined by the geology and the geography of the watershed. Characteristic drainage patterns are features which can be readily distinguished on aerial photographs and can be interpreted very rapidly.

Drainage density has a strong influence on both the spatial and temporal response of a watershed to a given precipitation event. If a watershed is well covered by a pattern of interconnected drainage channels, and the overland flow time is relatively short, the watershed will respond more rapidly than if it were sparsely drained and flow time was relatively long. The mean velocity of water is normally lower for overland flow than it is for flow in a well defined natural channel. High drainage density increases the response of a watershed leading to higher peak discharges and shorter hydrographs for a given precipitation event, Figure 14d.

Drainage density has minimal effect on the stage-discharge relationship for a particular section of drainage channel. It does, however, have an effect on the total volume of runoff since some of the abstraction processes are directly related to how long the rainfall excess exists as overland flow. Therefore, the lower the density of drainage, the lower will be the volume of flow from a given precipitation event.

Drainage density has an indirect effect on the frequency of discharges of given magnitudes. By strongly influencing the response of a given watershed to any precipitation input, the drainage density determines in part the frequency response. The higher the drainage density, the more closely related the resultant runoff frequency will be to that of the corresponding precipitation event.

### 2.4.6 Channel Length

Channel length plays an important role in several runoff characteristics. The longer the channel the more time it takes for water to be conveyed from the beginning of the channel to the outlet. Consequently, if all other factors are the same, a watershed with a longer channel length will have a slower response to a given precipitation input than a watershed with a shorter channel length. As the hydrograph travels along a channel, it is attenuated and extended in time due to the effects of channel storage and hydraulic roughness. As shown in Figure 14e, longer channels result in lower peak discharges and longer hydrographs.

The frequency of discharges of given magnitudes will also be influenced by channel length. As was the case for drainage density, channel length is an important parameter in determining the response time of a watershed to precipitation events of given frequency. However, channel length may not remain constant with discharges of various magnitudes. In the case of a wide flood plain where the main channel meanders appreciably, it is not unusual for the higher flood discharges to overtop the banks and essentially flow in a straight line in the flood plain, thus reducing the effective channel length.

The stage-discharge relationship and the total volume of runoff are practically independent of channel length. Volume, however, will be redistributed in time, similar in effect to storage but less pronounced.

### 2.4.7 Antecedent Moisture Conditions

As noted earlier, antecedent moisture conditions, which are the soil moisture conditions of the watershed at the beginning of a storm, affect the volume of runoff generated by a particular storm event. Runoff volumes are related directly to antecedent moistures. The lower the moisture in the ground at the beginning of precipitation, the lower will be the runoff; conversely, the higher the moisture content of the soil, the higher the runoff attributable to a particular storm.

### 2.4.8 Other Factors

There can be other factors within the watershed which determine the character of runoff. Examples are: extent and type of vegetation, the presence of channel modifications, and flood control structures. These factors modify the character of the runoff by either augmenting or negating some of the basin characteristics described above. It is important to recognize that all of the factors discussed exist concurrently within a given watershed, and their combined effects are very difficult to model and quantify.

## 2.5 Analysis of the Runoff Process

In Section 2.2 several key abstractions were described in general terms. The method by which the runoff process can be analyzed and the results used to obtain a hydrograph are illustrated in the following example. Figures 15a through 15h show the development of the flood hydrograph from a typical rainfall event.

### 2.5.1 Rainfall Input

Rainfall is randomly distributed in time and space and the rainfall experienced at a particular point can vary greatly. For simplification, consider the rainfall at only one point in space and assume that the variation of rainfall intensity with time can be approximated by discrete time periods of constant intensity. This simplification is illustrated in Figure 15a. The specific values of intensity and time are not important for this illustrative example since it shows only relative magnitudes and relationships. The rainfall, so arranged, is the input to the runoff process, and from this, the various abstractions must now be deleted.

### 2.5.2 Interception

Figure 15b illustrates the relative magnitude and time relationship for interception. When the rainfall first begins, the foliage and other intercepting surfaces are dry. As water adheres to these surfaces, a large portion of the initial rainfall is abstracted. This occurs relatively fast and once the initial wetting is complete, the interception losses quickly decrease to a lower, nearly constant value. The rainfall which has not been intercepted falls to the ground surface to continue in the runoff process.

### 2.5.3 Depression Storage

Figure 15c illustrates the relative magnitude of depression storage with time. Only the water which is in excess of that necessary to supply the interception is available for depression storage. This is the reason the depression storage curve begins at zero. The amount of water which goes into depression storage varies with differing land uses and soil types but the curve shown is representative. The smallest depressions are filled first and then the larger depressions are filled as time and the rainfall supply continue. The slope of the depression storage curve depends on the distribution of storage volume with respect to the size of depressions. There are usually many small depressions which fill rapidly and account for most of the total volume of depression storage. This results in a rapid peaking of storage with time as shown in Figure 15c. The large depressions take longer to fill and the curve gradually approaches zero when all the depression storage has been filled. If the rainfall input were less than the interception and depression storage, there would be no surface runoff.

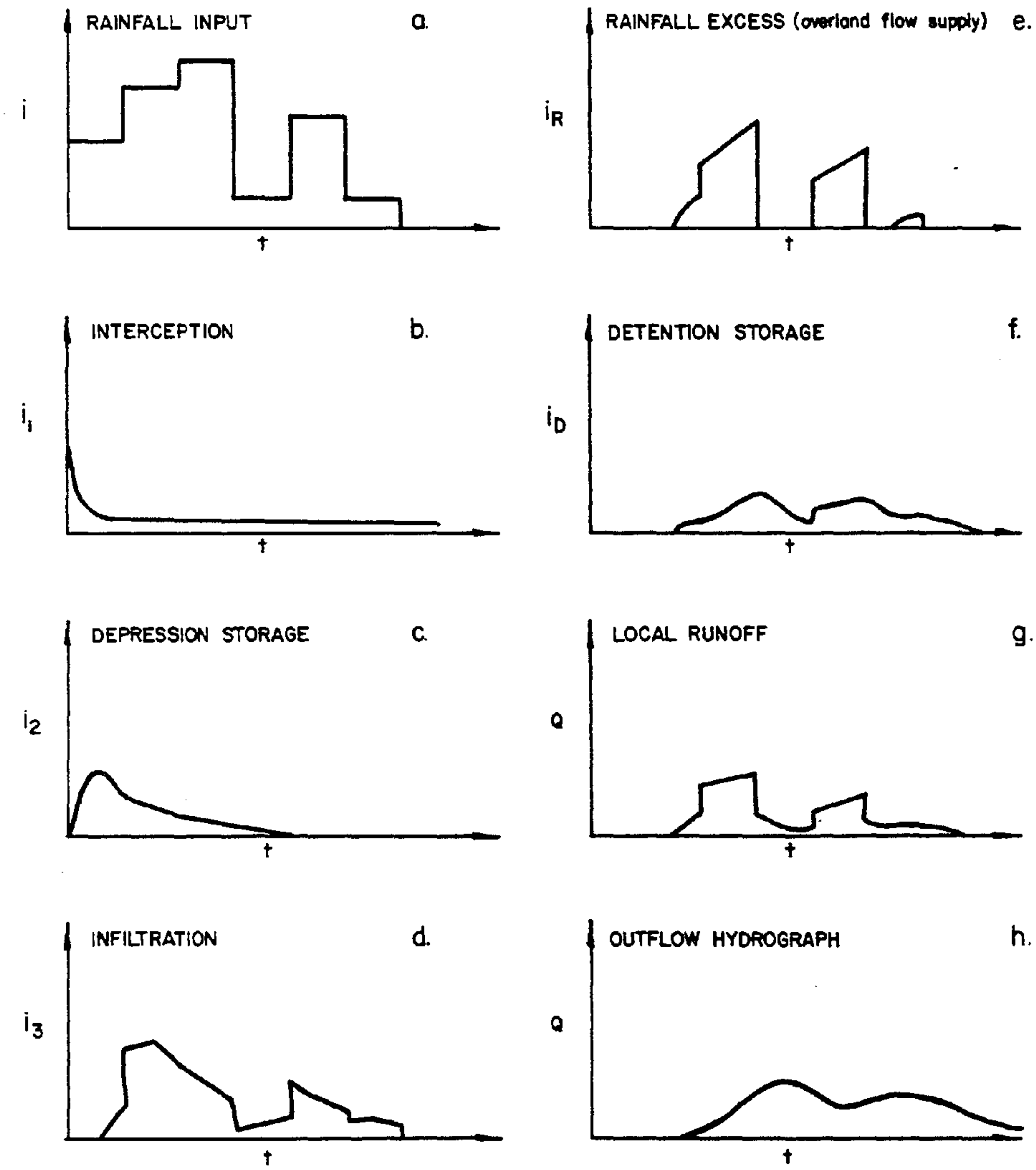



Figure 15. The Runoff Process

### 2.5.4 Infiltration

Infiltration is a complex process, and the rate of infiltration at any point in time depends on many factors as discussed in Section 2.2.3. The important point to be illustrated in Figure 15d is the time dependence of the infiltration curve. It is also important to note the behavior of the infiltration curve after the period of relatively low rainfall intensity near the middle of the storm event. The infiltration rate increases over what it was prior to the period of lower intensity. This is because the upper layers of the soil are drained at a rate which is independent of the rainfall intensity. The details of the process are not important but this phenomena should be recognized. Most deterministic models, including the Φ-Index method of estimating infiltration discussed in the later sections of this manual, do not model the infiltration process accurately in this respect.

### 2.5.5 Rainfall Excess

Only after interception, depression storage and infiltration have been satisfied is there an excess of water available to runoff from the land surface. As previously defined, this is the rainfall excess and is illustrated in Figure 15e. Note how this rainfall excess differs with the actual rainfall input, Figure 15a. The concept of excess rainfall is very important in hydrologic analyses. It is the amount of water available to runoff after the initial abstractions have been satisfied. Except for the losses that may occur during overland and channelized flow, it is the volume of water that flows by the outlet of a drainage basin. In other words, it should be very nearly equal to the volume under the hydrograph as defined in Section 2.3.4. The rainfall excess has a direct effect on the characteristics of the outflow hydrograph. It determines the magnitude of the peak flow, the time of flood and the shape of the hydrograph.

### 2.5.6 Detention Storage

As shown in Figure 15f, there is also a volume of water detained in temporary or detention storage. This volume is proportional to the local rainfall excess and is dependent on a number of other factors as mentioned in Section 2.2.6. Although all water in detention storage eventually leaves the basin, this requirement must be met before runoff can occur.

### 2.5.7 Local Runoff

Local runoff, illustrated in Figure 15g, is actually the residual of the rainfall input after all abstractions have been satisfied. It is similar in shape to the excess rainfall, Figure 15e, but is extended in time as the detention storage is depleted.

### 2.5.8 Outflow Hydrograph

Figure 15h illustrates the final outflow hydrograph from the watershed due to the local runoff hydrograph of Figure 15g. This final hydrograph is the cumulative effect of all the modifying factors which act on the water as it flows through drainage channels as discussed in Section 2.4. The total volume of water contained under the hydrographs of Figures 15g and 15h and the rainfall excess, Figure 15e are the same, although the outflow hydrograph's position in time is modified due to channel slope, length, roughness and storage.

The processes which have been discussed in the previous sections all act simultaneously to transform the incoming rainfall from that shown in Figure 15a to the corresponding outflow hydrograph of Figure 15h. This example serves to illustrate the runoff process for a small local area. If the watershed is of appreciable size or if the storm is large, then areal and time variations and other factors add a new level of complexity to the problem.

# 3.0 HYDROLOGIC DATA

As a first step in a hydrologic study, it is desirable to identify the data needs as precisely as possible. These needs will depend on whether the project is preliminary and accuracy is not critical, or if the analysis is to be performed to obtain parameters for final design. If the purpose of the study is defined, it is usually possible to select a method of analysis for which the type and amount of data can be readily determined. These data may consist of details of the watershed such as maps, topography, and land use, records of precipitation for various storm events, and information on annual or partial peak flows or continuous streamflow records. Depending on the size and scope of the project, it may even be necessary to seek out historical data on floods in order to better define the streamflow record.

If data needs are clearly identified, the effort necessary for its collection and compilation can be tailored to the importance of the project. Often, a well thought out data collection program generally leads to a more orderly and efficient analysis. It should be remembered, however, that data needs vary with the method of analysis, and that there is no single method applicable to all design problems.

Once data needs have been properly defined the next step is to identify possible sources of data. Past experience is the best guide as to which sources of data are likely to yield the required information. There is no substitute for actually searching through all the possible sources of data as a means of becoming familiar with the types of data available. This experience will pay dividends in the long run even if the data required for a particular study is not available in the researched sources. By acquainting the designer with the data that are available and the procedures necessary to access the various data sources the time required for subsequent data searches can often be significantly reduced.

## 3.1 Collection and Compilation of Data

Most of the data and information necessary for the design of highway stream crossings are obtained from some combination of the following sources:

1. Site investigations and field surveys
2. Files of federal agencies such as the National Weather Service, U.S. Geological Survey, Soil Conservation Service, among others
3. Files of state and local agencies such as State Highway Departments, Water Agencies and various planning organizations
4. Other published reports and documents

Certain types of data are needed so frequently, that some Highway Departments have compiled them into a single document, typically a Drainage Manual. Having data available in a single source greatly speeds up the retrieval of needed data and also helps to standardize the hydrologic analysis of highway drainage design.

### 3.1.1 Site Investigations and Field Surveys

It must be remembered that every problem is unique and that reliance on rote application of a standardized procedure, without due appreciation of the characteristics of the particular site is risky at best. A field survey or site investigation should always be conducted except for the most preliminary analysis or trivial designs. The field survey is one of the primary sources of hydrologic data.

The need for a field survey which appraises and collects site specific hydrologic and hydraulic data cannot be overstated. The value of such a survey has been well documented by the American Association of State Highway and Transportation Officials (AASHTO) Highway Drainage Guidelines and in Federal Highway Administration (FHWA) policy documents and guidelines.

Typical data which are collected during a field survey include highwater marks, assessments of the performance of nearby drainage structures, assessments of stream stability and scour potential, location and nature of important physical and cultural features which could affect or be affected by the proposed structure, significant changes in land use from those indicated on available topographic maps, and other equally important and necessary items of information which could not be obtained from other sources.

In order to maximize the amount of data that results from a field site survey the following should be standard procedure:

1. Individual in charge of the drainage aspects of the field site survey should have a general knowledge of drainage design

2. Data collected should be well documented with written reports and photographs

3. Field site survey should be well planned and a systematic approach employed to maximize efficiency and reduce wasted effort

The Federal-Aid Highway Programs Manual, 1974, contained a checklist for drainage studies and reports. In 1982, revised guidance which replaces the original checklist was issued in accordance with Executive Order 11988 for use in conducting studies for the evaluation of highway encroachments on flood plains. The updated version of this guidance is reproduced in its entirety in Appendix B. This checklist is intended as a guideline of items normally considered for inclusion in studies and reports. However, it is not all-inclusive and is not meant as a substitute for careful recording and documenting of other important and/or unusual physical and hydrologic features observed by the site inspection team.

The field survey should be performed by highway personnel responsible for the actual design or can be performed by the location survey team if they are well briefed and well prepared. Though the site survey is considered of paramount importance, it is but one data source and must be augmented by additional information from other reliable sources.

### 3.1.2 Sources of Other Data

An excellent source of data are the records and reports which other federal, state and municipal public works agencies have published or maintain. Many such agencies have been active in drainage design and construction and have data which can be very useful for a particular highway project. The designer who is responsible for highway drainage design should become familiar with the various agencies which are, or have been, active in an area. A working relationship with these agencies should be established, either formally or informally, to exchange data for mutual benefit.

To aid in identifying possible sources of information from a few of the more active Federal agencies a list of addresses and telephone numbers have been compiled and are included in Appendix C. The agencies listed are the U.S. Army Corps of Engineers, the U.S. Geological Survey, the U.S. Soil Conservation Service, the U.S. Forest Service, the Bureau of Reclamation, the Tennessee Valley Authority, the Federal Emergency Management Agency, and the Environmental Protection Agency.

Historical records or accounts are another source of data which should never be overlooked by the highway designer. Floods are noteworthy events and very often the occurrence of a flood and specific information such as high water elevations are recorded. Sources of such information include newspapers, magazines, State historical societies or universities, and publications by several Federal agencies. Recent storms or flood events of historic proportion have been very thoroughly documented by the U.S. Geological Survey (USGS), the Corps of Engineers and the National Weather Service (NWS). The publications of interested sources can be used to define storm events that may have occurred in the area of concern and their information should be noted.

The sources of information and data referred to in the preceeding paragraphs may provide hydrologic data in a form suitable for analysis by the highway designer. There are other sources of data which will provide information of a more basic nature. An example is the data available from the USGS for the network of stream gaging stations which this agency maintains throughout the country. This type of information is the basis for any hydrologic study and the highway designer needs to know where to find it. The information categories are:

1. Streamflow records
2. Precipitation records
3. Soil types

4. Land use

5. Other types of basic data needed for hydrologic analysis

#### 3.1.2.1 Streamflow Data

The major source of streamflow information is the USGS, an agency charged with collecting and disseminating this data. The USGS collects data at approximately 16,000 stream-gaging stations nationwide. This data is compiled by the USGS and is published in Water Supply Papers and also added to a data base called the Water Data Storage and Retrieval System or WATSTORE. WATSTORE is accessible through the USGS District Offices, a list of which are included in Appendix C.

WATSTORE contains a Peak Flow File Retrieval Program, J980, which provides pertinent characteristics of the station and drainage area and a listing of both peak annual and secondary floods by Water Year (October through September). Table 2 is an example J980 output for Station 08181500, Medina River at San Antonio. The annual peaks from Program J980 are used in conjunction with the frequency analysis program available through WATSTORE. The Peak flow data of Table 2 are also used subsequently in Section 4.0 to illustrate various standard frequency distributions and as input to a frequency analysis program contained in WATSTORE.

Also, the Corps of Engineers and the Bureau of Reclamation collect streamflow data. These two agencies along with the USGS together account for about 90 percent of the stream flow data that are available in the United States. Other sources of data are local utility companies, water-intensive industries and academic or research institutions.

Streamflow data is one of the types of data referenced by the National Water Data Exchange (NAWDEX). NAWDEX is a nationwide confederation of water-oriented organizations working together to improve access to water data. Their primary objective is to assist users of water data in the identification, location, and acquisition of needed data. (NAWDEX will be described more fully later in this section.)

#### 3.1.2.2 Precipitation Data

The major source of precipitation data is the National Weather Service (NWS). Precipitation and other measurements are made at approximately 20,000 locations each day. The measurements are fed through the Weather Service Forecast Offices (WSFO) which serve each of the 50 States, and Puerto Rico.

Each WSFO uses this data and information obtained via satellite and other means, to forecast the weather for its area of responsibility. In addition to the WSFO's, the Weather Service maintains a network of River Forecast Centers

# Table 2. Sample Output, USGS Program J980 for Peak Flow Retrieval

```
WATSTORE PEAK FLOW FILE RETRIEVAL PGM. J980 - RUN DATE : 19 JUL 84  14.57.58
PROGRAM LAST REVISED :  3 OCT 83 18.25.23

A PASSWORD WAS SUPPLIED ON EXEC CARD


*** EXPLANATION OF PEAK DATA CODES *****

    DISCHARGE QUALIFICATION CODES:

      1...DISCHARGE IS A MAXIMUM DAILY AVERAGE
      2...DISCHARGE IS AN ESTIMATE
      3...DISCHARGE AFFECTED BY DAM FAILURE
      4...DISCHARGE LESS THAN INDICATED VALUE, WHICH IS MINIMUM RECORDABLE DISCHARGE AT THIS SITE
      5...DISCHARGE AFFECTED TO UNKNOWN DEGREE BY REGULATION OR DIVERSION
      6...DISCHARGE AFFECTED BY REGULATION OR DIVERSION
      7...DISCHARGE IS AN HISTORIC PEAK
      8...DISCHARGE ACTUALLY GREATER THAN INDICATED VALUE
      9...DISCHARGE DUE TO SNOWMELT, HURRICANE, ICE-JAM OR DEBRIS DAM BREAKUP
      A...YEAR OF OCCURRENCE IS UNKNOWN OR NOT EXACT
      B...MONTH OR DAY OF OCCURRENCE IS UNKNOWN OR NOT EXACT
      C...ALL OR PART OF THE RECORD AFFECTED BY URBANIZATION, MINING, AGRICULTURAL CHANGES, CHANNELIZATION, OR OTHER
      D...BASE DISCHARGE CHANGED DURING THIS YEAR
      E...ONLY ANNUAL MAXIMUM PEAK AVAILABLE FOR THIS YEAR

    GAGE HEIGHT QUALIFICATION CODES:

      1...GAGE HEIGHT AFFECTED BY BACKWATER
      2...GAGE HEIGHT NOT THE MAXIMUM FOR THE YEAR
      3...GAGE HEIGHT AT DIFFERENT SITE AND/OR DATUM
      4...GAGE HEIGHT BELOW MINIMUM RECORDABLE ELEVATION
      5...GAGE HEIGHT IS AN ESTIMATE
      6...GAGE DATUM CHANGED DURING THIS YEAR


*** NOTES *****

BASE DISCHARGE (IF REPORTED) MAY NOT BE EFFECTIVE FOR ENTIRE PERIOD OF RECORD; CURRENT VALUE USED.

GAGE DATUM (IF REPORTED) MAY NOT BE EFFECTIVE FOR ENTIRE PERIOD OF RECORD; CURRENT VALUE USED.



RETRIEVAL SPECIFICATIONS FOR REQUEST NUMBER 01 ARE AS FOLLOWS:
M CARD:  M                                                                     01
PEAK FLOW RETRIEVAL NUMBER 01 IS FOR ALL WATER YEARS
THE FOLLOWING HAVE BEEN REQUESTED:
.....LONG FORMAT PRINTOUT
.....STANDARD RECORD FORMAT


NUMBER OF SITES RETRIEVED:       1
NUMBER OF RECORDS RETRIEVED:      43



END OF RETRIEVAL PROCESSING
```

Table 2. Sample Output, USGS Program J980 for Peak Flow Retrieval (Continued)

STATION 08181500 MEDINA RIVER AT SAN ANTONIO, TEX.

| | | | | | |
|---|---|---|---|---|---|
| AGENCY: | USGS | STATION LOCATOR | DRAINAGE AREA: | 1317.00 | SQ MI |
| STATE: | 48 | LAT. LONG. | CONTRIBUTING | | |
| COUNTY: | 029 | | DRAINAGE AREA: | | SQ MI |
| DISTRICT: | 48 | 291514 0982820 | GAGE DATUM: | 439.00 | (NGVD) |
| | | | BASE DISCHARGE: | 1500.00 | CFS |

| WATER YEAR | DATE | PEAK DISCHARGE (CFS) | DISCHARGE CODES | GAGE HEIGHT (FT) | GAGE HT CODES | HIGHEST SINCE | MAX GAGE HEIGHT (FT) | DATE | GAGE HT CODES | NUMBER OF PARTIAL PEAKS |
|---|---|---|---|---|---|---|---|---|---|---|
| 1940 | 06/30/40 | 2540.00 | 6 | 15.97 | | | | | | 0 |
| 1941 | 02/02/41 | 6890.00 | 6 | 22.93 | | | | | | 2 |
| | 11/01/40 | 2350.00 | | | | | | | | |
| | 04/28/41 | 3140.00 | | | | | | | | |
| 1942 | 09/05/42 | 17500.00 | 6 | 30.92 | | | | | | 3 |
| | 07/05/42 | 3100.00 | | | | | | | | |
| | 09/08/42 | 7000.00 | | | | | | | | |
| | 09/09/42 | 5050.00 | | | | | | | | |
| 1943 | 10/18/42 | 12100.00 | 6 | 27.20 | | | | | | 1 |
| | 10/04/42 | 3040.00 | | | | | | | | |
| 1944 | 08/28/44 | 2000.00 | 6 | 13.33 | | | | | | 0 |
| 1945 | 02/12/45 | 3540.00 | 6 | 16.96 | | | | | | 2 |
| | 12/05/44 | 2090.00 | | | | | | | | |
| | 01/18/45 | 2930.00 | | | | | | | | |
| 1946 | 08/29/46 | 31800.00 | 6 | | | | | | | 1 |
| | 09/27/46 | 24800.00 | | | | | | | | |
| 1947 | 10/09/46 | 1470.00 | 6 | 12.57 | | | | | | 0 |
| 1948 | 08/27/48 | 2050.00 | 6 | 14.58 | | | | | | 0 |
| 1949 | 06/26/49 | 17400.00 | 6 | 30.79 | | | | | | 1 |
| | 04/25/49 | 2920.00 | | | | | | | | |
| 1950 | 10/25/49 | 5660.00 | 6 | 21.67 | | | | | | 0 |
| 1951 | 05/16/51 | 2150.00 | 6 | 14.92 | | | | | | 0 |
| 1952 | 09/12/52 | 801.00 | 6 | 9.11 | | | | | | 0 |
| 1953 | 09/04/53 | 4960.00 | 6 | 20.79 | | | | | | 1 |
| | 09/01/53 | 2800.00 | | | | | | | | |
| 1954 | 04/08/54 | 865.00 | 6 | 9.53 | | | | | | 0 |
| 1955 | 02/06/55 | 1200.00 | 6 | 11.35 | | | | | | 0 |
| 1956 | 09/01/56 | 1750.00 | 6 | 16.37 | | | | | | 0 |
| 1957 | 04/29/57 | 5180.00 | 6 | 22.83 | | | | | | 6 |
| | 10/19/56 | 2290.00 | | 18.30 | | | | | | |
| | 04/20/57 | 2130.00 | | 17.76 | | | | | | |
| | 05/19/57 | 1950.00 | | 15.81 | | | | | | |
| | 05/28/57 | 3240.00 | | 19.37 | | | | | | |
| | 06/02/57 | 3090.00 | | 19.05 | | | | | | |
| | 09/25/57 | 2100.00 | | 16.33 | | | | | | |
| 1958 | 05/03/58 | 9220.00 | 6 | 27.79 | | | | | | 4 |
| | 10/22/57 | 2250.00 | | 16.80 | | | | | | |
| | 02/22/58 | 2470.00 | | 17.47 | | | | | | |
| | 09/21/58 | 6000.00 | | 24.00 | | | | | | |
| | 09/24/58 | 2250.00 | | 16.75 | | | | | | |
| 1959 | 10/30/58 | 3350.00 | 6 | 19.56 | | | | | | 0 |
| 1960 | 10/04/59 | 3200.00 | 6 | 19.33 | | | | | | 0 |
| 1961 | 07/23/61 | 3050.00 | 6 | 17.92 | | | | | | 2 |
| | 10/29/60 | 1750.00 | | 16.16 | | | | | | |
| | 06/20/61 | 1630.00 | | 14.23 | | | | | | |
| 1962 | 10/26/61 | 3960.00 | 6 | 19.57 | | | | | | 0 |

Table 2. Sample Output, USGS Program J980 for Peak Flow Retrieval (Continued)

| | | | | | |
|---|---|---|---|---|---|
| 1963 | 09/14/63 | 890.00 | 6 | 10.22 | 0 |
| 1964 | 10/23/63 | 2140.00 | 6 | 15.84 | 2 |
| | 03/19/64 | 1570.00 | | 13.80 | |
| | 06/17/64 | 1960.00 | | 15.74 | |
| 1965 | 05/18/65 | 5430.00 | 6 | 23.52 | 3 |
| | 10/26/64 | 1560.00 | | 13.97 | |
| | 11/05/64 | 3630.00 | | 20.75 | |
| | 02/10/65 | 1720.00 | | 14.80 | |
| 1966 | 12/04/65 | 2160.00 | 6 | 16.68 | 0 |
| 1967 | 09/22/67 | 5480.00 | 6 | 23.56 | 0 |
| 1968 | 01/18/68 | 13100.00 | 6 | 28.56 | 2 |
| | 01/21/68 | 8940.00 | | 24.67 | |
| | 05/12/68 | 3220.00 | | 19.44 | |
| 1969 | 05/05/69 | 2730.00 | 6 | 18.32 | 3 |
| | 05/13/69 | 1630.00 | | 14.36 | |
| | 06/05/69 | 1500.00 | | 13.73 | |
| | 08/28/69 | 2590.00 | | 17.98 | |
| 1970 | 05/15/70 | 3360.00 | 6 | 19.71 | 1 |
| | 05/31/70 | 1830.00 | | 15.23 | |
| 1971 | 08/04/71 | 2950.00 | 6 | 18.88 | 2 |
| | 08/06/71 | 2660.00 | | 18.16 | |
| | 08/15/71 | 2680.00 | | 18.20 | |
| 1972 | 05/08/72 | 6360.00 | 6 | 23.15 | 3 |
| | 10/22/71 | 3300.00 | | 19.60 | |
| | 05/11/72 | 2200.00 | | 16.63 | |
| | 05/13/72 | 3320.00 | | 19.65 | |
| 1973 | 07/17/73 | 31900.00 | 6 | 43.59 | 5 |
| | 04/16/73 | 2460.00 | | 17.52 | |
| | 04/18/73 | 2370.00 | | 17.23 | |
| | 06/26/73 | 2250.00 | | 16.82 | |
| | 09/17/73 | 9600.00 | | 26.09 | |
| | 09/27/73 | 16800.00 | | 32.56 | |
| 1974 | 08/31/74 | 9680.00 | 6 | 26.18 | 4 |
| | 10/12/73 | 1700.00 | | 14.66 | |
| | 10/14/73 | 2450.00 | | 17.49 | |
| | 10/16/73 | 4560.00 | | 21.40 | |
| | 09/08/74 | 8050.00 | | 24.72 | |
| 1975 | 02/04/75 | 4130.00 | 6 | 20.86 | 2 |
| | 05/26/75 | 2110.00 | | 16.31 | |
| | 06/08/75 | 1940.00 | | 15.69 | |
| 1976 | 05/08/76 | 7510.00 | 6 | 23.48 | 3 |
| | 04/19/76 | 7280.00 | | 23.20 | |
| | 05/13/76 | 3040.00 | | 17.03 | |
| | 05/26/76 | 2900.00 | | 16.56 | |
| 1977 | 09/13/77 | 4620.00 | 6 | 21.46 | 3 |
| | 10/05/76 | 1530.00 | | 12.75 | |
| | 10/30/76 | 4390.00 | | 19.68 | |
| | 04/20/77 | 3980.00 | | 21.10 | |
| 1978 | 08/04/78 | 9440.00 | 6 | 25.95 | 0 |
| 1979 | 06/01/79 | 4750.00 | 6 | 21.61 | 0 |
| 1980 | 09/11/80 | 1980.00 | | 15.84 | 0 |
| 1981 | 06/14/81 | 14500.00 | 6 | 29.04 | 0 |
| 1982 | 05/17/82 | 8160.00 | | 23.30 | 0 |

END OF PROGRAM J980.

(RFC). These River Forecast Centers prepare river and flood forecasts for about 2500 communities. These two organizational units of the National Weather Service are an excellent source of data and information.

A list of the six Regional National Weather Service Offices is included in Appendix C to assist the highway engineer in obtaining data from the NWS. The National Weather Service is a part of the National Oceanic and Atmospheric Administration (NOAA), and the data collected by the NWS and other organizations within NOAA are sent to the Environmental Data and Information Service (EDIS). The EDIS is charged with the responsibility of collecting, processing, and disseminating environmental data, and it is an excellent source of basic data with which the designer should be familiar. An address for the Environmental Data and Information Service is included in Appendix C.

#### 3.1.2.3 Soil Type Data

Information on the type of soil which is characteristic of a particular region is often needed as a basic input in hydrologic evaluations. The major source of soil information is the Soil Conservation Service which is actively engaged in the classification and mapping of the soils across the country. Soil maps have been or are being prepared for most of the counties in the country. The highway designer should contact the SCS or county extension service to determine the availability of this data. A list of addresses for SCS State offices has been included in Appendix C.

#### 3.1.2.4 Land Use Data

Land use data is available in different forms such as: topographic maps, aerial photographs, zoning maps, and Landsat images. These different forms of data are available from many different sources such as State, Regional or municipal planning organizations, the U.S. Geological Survey and the Natural Resource Economic Division, Water Branch, of the Department of Agriculture. The highway designer should become familiar with the various planning or other land use related organizations within his geographic area of interest, and the types of information which they collect, publish or record.

#### 3.1.2.5 Miscellaneous Basic Data

Aerial photographs are an excellent source of hydrologic information and the Soil Conservation Service and State Highway Departments are good sources of such photographs. Another source of aerial photographs is the USGS, through the National Cartographic Information Center (NCIC). The NCIC operates a national information service for U.S. cartographic and geographic data. They provide access to a number of useful cartographic and photographic products. A few of these products are land-use and land cover maps, orthophotoquads (black and white photo images in standard USGS quadrangle format), aerial photographs covering the entire country, Landsat images (both standard and computer enhanced), photo indexes showing the prints available for standard USGS quadrangles, and many other services and products too numerous to list. The address of the NCIC is included in Appendix C.

Other types of basic data which might be needed for a hydrologic analysis include data on infiltration, evaporation, geology, snowfall, solar radiation, and oceanography. Sources of these types of data are scattered and the designer must rely upon his past experience or the experience of others to help locate them. (In order to utilize the combined experience of others it is wise to develop strong working relationships with other professionals active in the same geographic area). The Environmental Data and Information Service (EDIS) is a good starting point for the collection of miscellaneous types of data.

3.1.2.6 National Water Data Exchange

As can be seen from the discussion above there are a number of different sources of hydrologic data. In fact there are so many that just keeping track of them is an enormous job. It is for this reason that NAWDEX (National Water Data Exchange) was founded. The primary objective of NAWDEX is to assist users of water data in the identification, location, and acquisition of needed data. NAWDEX became operational in 1976 and currently provides relatively easy access to vast amounts of water related data.

NAWDEX maintains two major files. The first is the WATER DATA SOURCES DIRECTORY which identifies organizations which collect water data, locations within these organizations from which water data may be obtained, the geographic area in which the organization collects water data, the types of water data collected and available, and alternate sources from which the organization's water data may be obtained. Information has been compiled for more than 660 organizations, and more will be added on a continuing basis. The second major file is the MASTER WATER DATA INDEX which provides a nationwide indexing service of water data collection sites. Over 375,000 sites are indexed by geographic locations, the data-collecting organization, the types of data available, the period of time for which the data are available, the major water-data parameters for which data are available, the frequency of measurement and the media in which the data are stored. The WATER DATA SOURCE DIRECTORY and the MASTER WATER DATA INDEX contain common identifiers which allow them to be used together. For example, the MASTER WATER DATA INDEX may be used to identify water data available in a geographic area and the WATER DATA SOURCES DIRECTORY may then be used to obtain the names and addresses of organizations from which the data may be obtained.

NAWDEX is maintained by the U.S. Geological Survey and access to NAWDEX is through a nationwide network of 60 Assistance Centers. A current directory containing the names, addresses and telephone numbers of all Assistance Centers is available from the NAWDEX Program Office. The address for the NAWDEX Program Office is included in Appendix C.

Using the agencies mentioned above, the highway designer should have ample sources to begin collecting the specific data needed. However, there is another source of information which the designer will need. This is the broad collection of general information sources which are invaluable aids in hydrologic analyses. Among them are general references such as textbooks, drainage or hydrology manuals of State or Federal agencies, atlases, special reports and technical publications, journals of professional societies and university publications. It is essential that an adequate hydrologic library be established and maintained so that the wealth of available information is easily accessed. It is equally important that a systematic effort be made to keep abreast of new developments and methods which could improve the accuracy or efficiency of hydrologic analyses.

## 3.2 Adequacy of Data

Once the needed hydrologic data has been collected, the next step is to compile the data into a usable format. The designer must ascertain whether the data contain inconsistencies or other unexplained anomalies which might lead to erroneous calculations or results. The main reason for analyzing the data is to draw all of the various pieces of collected information together, and to fit them into a comprehensive and accurate representation of the hydrology at a particular site.

Experience, knowledge, and judgment are an important part of data evaluation. It is in this phase that reliable data must be separated from that which is not so reliable and historical data combined with that obtained from measurements. The data must be evaluated for consistency and to identify any changes from established patterns. At this time, any gaps in the data record should either be justified or filled in if possible. Some of the methods and techniques discussed later in this manual are useful for this purpose. The methods of statistics can be of great value in data analysis, but it must be emphasized that an underlying knowledge of hydrology is essential for prudent and meaningful application of these statistical methods. It is also helpful to review previous studies and reports for types and sources of data, how the data were used, and any indications of accuracy and reliability. Historical data should be reviewed to determine whether significant changes have occurred in the watershed that might affect its hydrology and whether these data can be used to possibly improve or extend the period of record.

Basic data, such as streamflow and precipitation, need to be evaluated for hydrologic homogeneity and summarized before use. Maps, aerial photographs, Landsat images, and land use studies should to be compared with one another and with the results of the field survey so any inconsistencies can be resolved. General references should be consulted to help define the hydrologic character of the site or region under study, and to aid in the analysis and evaluation of data.

The results of this type of data evaluation should provide a description of the hydrology of the site within the allotted time and the resources committed to this effort. Obviously, not every project will be the same, but the designer must adequately define the parameters necessary to design the needed drainage structures to the required reliability.

### 3.3 Presentation of Data and Analysis

If the data needs have been clearly identified, the results of the analysis can be readily summarized in an appropriate manner and quickly used in the selected method of hydrologic analysis. The data needs of each method are different so no single method of presenting the data will be applicable to all situations. However, there are a few methods of hydrologic analysis which are used so frequently that standardized formats are appropriate. These will be illustrated with examples in subsequent sections of this report.

#### 3.3.1 Documentation

The results of the data collection and data evaluation phases must be documented in order to:

1. Provide a record of the data itself
2. Provide references to data which have not been incorporated into the record because of its volume or for other reasons
3. Provide references for the methods of data analysis used
4. Document assumptions, recommendations and conclusions
5. Present the results in a form compatible with the analytical method utilized
6. Index the data and analysis for ease of retrieval
7. Provide support of expenditures of public funds by highway agencies

It is always sound engineering practice to thoroughly document the work. The format, or method, used to document the collected data or subsequent analysis should be standardized. In this way, those unfamiliar with a specific project may readily refer to the needed information. This is especially important in those States where there are several different offices or districts performing hydrologic analyses and design. It is important that all of the data collected is either included in the documentation or adequately referenced so that it may be quickly retrieved. This is true, whether or not the data were used in the subsequent analysis, since it could be very useful in a future study.

It is important that all data analyses be presented in the documentation. If several different methods were used, then each analysis should be reported and documented, even if the results were not included in the final recommendations. Pertinent comments as to why certain results were either discounted or accepted should be a part of the documentation.

All methods used should be referenced to a source such as a State drainage manual, textbook or other publication. The edition, date and author (if known) of each reference should be included. It is helpful to include a notation as to where a particular reference should be consulted. It is also helpful to identify where a particular reference is available.

Perhaps the most important part of the documentation is the recording of assumptions, conclusions and recommendations which are made during or as a result of the collection and analysis of the data. Since hydrology is not an exact science, it is impossible to adequately collect and analyze hydrologic data without using judgment and making some assumptions. By recording these subjective judgments, the designer not only provides a more detailed and valuable record of his work, but the documentation will prove invaluable to younger, less experienced, personnel who can be educated by exposure to the judgment and experience of their peers.

### 3.3.2 Indexing

The value of the data collected and its subsequent analysis is greatly enhanced if the data can be retrieved easily and used again in the future. In order for others to find previous studies which contain usable information, it is necessary to positively identify and physically locate the studies. This process is facilitated by a well thought out system of indexing the studies.

One of the best sources of data is the project files of the given Highway Department. Highway Departments have been studying, designing and constructing drainage structures for many years. The wealth of information which has been gathered and documented during previous work should be consulted routinely whenever a new project is studied or designed.

In order to be of use, it is important that the highway project records and files be cross referenced to facilitate their use as a data base for hydrologic studies. Frequently, project records are filed only by a project number which is based on the source of financing and route number. This often makes it difficult to retrieve the needed data. Some method of cross-referencing, which is keyed to a hydrologic index such as the name of a river basin or a hydrologic unit map number, is desirable. The hydrologic unit map number system was developed by the U.S. Geological Survey and utilizes a code consisting of from two to eight digits based on four levels of classification. The first level divides the United States into 21 major geographic regions and contains either a major river basin or the combined drainage areas of several rivers. The second level divides the 21 regions into 222 planning subregions, each including either the area drained by a river system, a reach of river and its tributaries, or a closed basin/s or groups of streams forming a coastal drainage area. The third level subdivides the planning subregions into 352 accounting units which are used in managing the National Water Data Network. The fourth level is the cataloging unit which represents all or part of a surface drainage area or distinct hydrologic feature. There are approximately 2,150 cataloging units in the Nation. An example of a hydrologic unit code is 01080204, where

01 - region
0108 - planning subregion
010802 - accounting unit
01080204 - cataloging unit

USGS Cirular 848A provides a map of all the regions, planning regions and accounting units in the United States and a list of all hydrologic unit codes including State and outlying areas. This hydrologic unit code is identical to that used to define gaging stations; for example, the code for the Medina River at San Antonio is given as 08181500 in Table 2.

If a system of documentation and indexing, such as that described above, is implemented and maintained for several years, then the most valuable source of hydrologic data may always be the files of one's own Highway Department.

## 4.0 FREQUENCY ANALYSIS FOR SITES WITH ADEQUATE DATA

The estimation of peak discharges of various recurrence intervals is one of the most common problems faced by highway engineers when designing for highway drainage structures. The problem can be divided into two categories:

1. Gaged sites - the site is at or near a gaging station and the streamflow record is fairly complete and of sufficient length to be used to provide estimates of peak discharges;

2. Ungaged sites - the site is not near a gaging station and no streamflow record is available.

Sites which are located at or near a gaging station but which have incomplete or very short records represent special cases. For these situations, peak discharges are estimated either by supplementing or transposing data and treating them as gaged sites; or by using regression equations or other synthetic methods applicable to ungaged sites.

Depending on the availability of data for a given site, the specified preference for the method by which peak flows are determined is as follows:

1. Statistical analysis of gaged data

2. U.S. Geological Survey regional or other regression equations for ungaged watersheds

3. Other synthetic methods including the Index Flood method and the Rational Formula

This section of the manual is concerned primarily with the statistical analysis of gaged data. Appropriate solution techniques are presented and the assumptions and limitations of each are discussed. Regional regression equations and other synthetic methods applicable to ungaged sites are covered in Section 5.0.

### 4.1 Basins with Adequate Data

The U.S. Geological Survey (USGS) is the leading agency in the collection of flood data and the maintenance of systematic peak discharge information. These data are reported in USGS Water Supply Papers, Annual Surface Water Records and computer files. Other federal, state, and local agencies identified in Appendix C maintain annual peak flow records which are available in published and unpublished form.

Analysis of gaged data permits an estimate of the peak discharge in terms of its probability or frequency of occurrence at a given site. This is done by statistical methods provided sufficient data are available at the site to permit a meaningful statistical analysis to be made. Water Resources Council

Bulletin 17B, 1981, suggests at least 10 years of record are necessary to warrant a statistical analysis by methods presented therein. The USGS at one time recommended that the period of record should be at least one-half the frequency of the design flow. In other words, if a 50-year design storm is desired, the period of record should be at least 25 years long. Based on further analyses and experience and the recognition that many stations do not have sufficient records, the USGS in 1973, relaxed this criteria to the following:

The 10-Year Design Period Flood needs 10 years of record.

The 25-Year Design Period Flood needs 15 years of record.

The 50-Year Design Period Flood needs 20 years of record.

The 100-Year Design Period Flood needs 25 years of record.

Although these guidelines were conservative, they have again been relaxed, and today, the USGS has no specified criteria for flood frequency determinations.

At some sites, there may be historical data on large floods prior to or after the period over which streamflow data were collected. This information can be collected from inquiries, newspaper accounts and from field surveys for highwater marks. Whenever possible, these data should be compiled and documented to improve on frequency estimates.

## 4.2 Statistical Character of Floods

This section serves to introduce the designer to those fundamental statistical concepts for the determination of peak flows. Statistical analysis is simply a systematic way of looking at data. Through the use of the methods of statistical analysis, the salient features of the data are quantified, thereby allowing the data to be generalized. It is also possible to use the methods of statistical analysis to predict future events based on the character of the past data.

Fundamental to statistical analysis are the concepts of populations and samples. A population which may be either finite or infinite is defined as the entire collection of all possible occurrences of a given quantity. An example of a finite population is the number of possible outcomes of the throw of the dice, a fixed number. An example of an infinite population is the number of different peak annual discharges possible for a given stream.

A sample is defined as part of a population. In all practical instances, hydrologic data are analyzed as a sample of an infinite population, and it is usually assumed that the sample is representative of its parent population. By representative, it is meant that the characteristics of the sample, such as its measures of central tendency and its frequency distribution, are the same as that of the parent population. There is an entire branch of statistics which deals with the inference of population characteristics and parameters from the characteristics of samples. The techniques of inferential statistics, which is the name of this branch of statistics, are very useful in the analysis of hydrologic data because samples are used to predict the characteristics of the populations. Not only will the techniques of inferential statistics allow estimates of the characteristics of the population from samples, but they also permit the evaluation of the reliability or accuracy of the estimates.

Once data has been collected it must be analyzed. The collection of data was covered in Section 3.0; the statistical analysis of the data is the subject of this section. There are several methods available for the analysis of data and these will be discussed below. For illustration, actual peak flow data will be analyzed by each of the methods presented.

Before analyzing data it is necessary that it be arranged in a systematic manner. Data can be arranged in a number of ways depending on the specific characteristics that are to be examined. An arrangement of data by a specific characteristic is called a distribution or a series. Some common types of data groupings are the following:

1. Magnitude
2. Time of Occurrence
3. Geographic Location

4.2.1 Arrangement by Magnitude

The most common arrangement of hydrologic data is by magnitude of the annual peak discharge. This arrangement is called an Annual Series. As an example of an Annual Series, the 29 annual peak discharges for Mono Creek near Vermilion Valley, California are listed and ordered according to magnitude and recurrence interval in Table 3.

Another method used in flood data arrangement is the Partial Duration Series, sometimes referred to as the Basic Stage Method. This procedure uses all peak flows above some base value. For example, the Partial Duration Series may consider all flows above the lowest annual peak flow as a base. Over a 20-year period of record, this may yield thirty or more floods compared to twenty floods in the Annual Series. Figure 16 illustrates a portion of the record for Mono Creek containing both the highest annual floods and other large secondary floods.

If these floods are ordered in the same manner as in an Annual Series, they can be plotted as illustrated in Figure 17. By separating out the peak annual flows, the two series can be compared as also shown in Figure 17 where it is seen that for a given order, m, the Partial Duration Series yields a

Table 3. Arrangement of Flood Data by Magnitude, Mono Creek, CA

Basin: Mono Creek - near Vermilion Valley, California, South Fork of San Joaquin River Basin

Location: Latitude 37° 22' 00", Longitude 118° 59' 20" - one mile downstream from lower end of Vermilion Valley and 6 miles downstream from North Fork.

Area: 92 square miles

Remarks: No diversion or regulation

Record: 1922 - 1950, 29 years (no data adjustments)

| Year | Peak annual Q~cfs | Q, arranged in order of magnitude | Order | Recurrence Interval $= \frac{\text{order}}{\text{Years of Record}}$ |
|---|---|---|---|---|
| 1922 | 1390 | 1760 | 1 | .0344 |
| 23 | 940 | 1440 | 2 | .0690 |
| 24 | 488 | 1420 | 3 | .1034 |
| 1925 | 1060 | 1420 | 4 | .1379 |
| 26 | 1030 | 1420 | 5 | .1724 |
| 27 | 1420 | 1390 | 6 | .2069 |
| 28 | 1110 | 1370 | 7 | .2414 |
| 29 | 750 | 1350 | 8 | .2759 |
| 1930 | 848 | 1230 | 9 | .3103 |
| 31 | 525 | 1210 | 10 | .3448 |
| 32 | 1420 | 1170 | 11 | .3793 |
| 33 | 1350 | 1130 | 12 | .4138 |
| 34 | 404 | 1110 | 13 | .4483 |
| 1935 | 1230 | 1100 | 14 | .4828 |
| 36 | 1060 | 1060 | 15 | .5172 |
| 37 | 1210 | 1060 | 16 | .5517 |
| 38 | 1760 | 1030 | 17 | .5862 |
| 39 | 540 | 988 | 18 | .6207 |
| 1940 | 1130 | 940 | 19 | .6552 |
| 41 | 1420 | 916 | 20 | .6897 |
| 42 | 1170 | 910 | 21 | .7241 |
| 43 | 1440 | 855 | 22 | .7586 |
| 44 | 855 | 848 | 23 | .7931 |
| 1945 | 1370 | 838 | 24 | .8276 |
| 46 | 910 | 750 | 25 | .8621 |
| 47 | 988 | 540 | 26 | .8966 |
| 48 | 938 | 525 | 27 | .9310 |
| 49 | 916 | 488 | 28 | .9655 |
| 1950 | 1100 | 404 | 29 | 1.0000 |

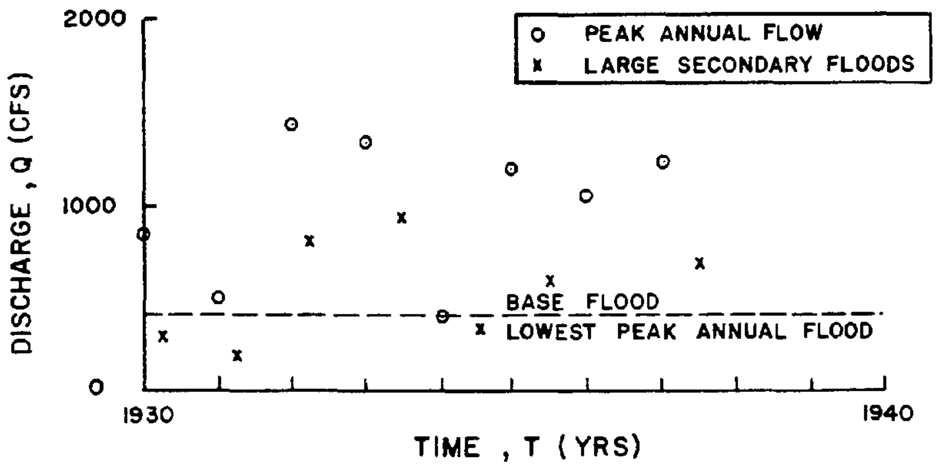



Figure 16. Peak Annual and Other Large Secondary Flows, Mono Creek, CA

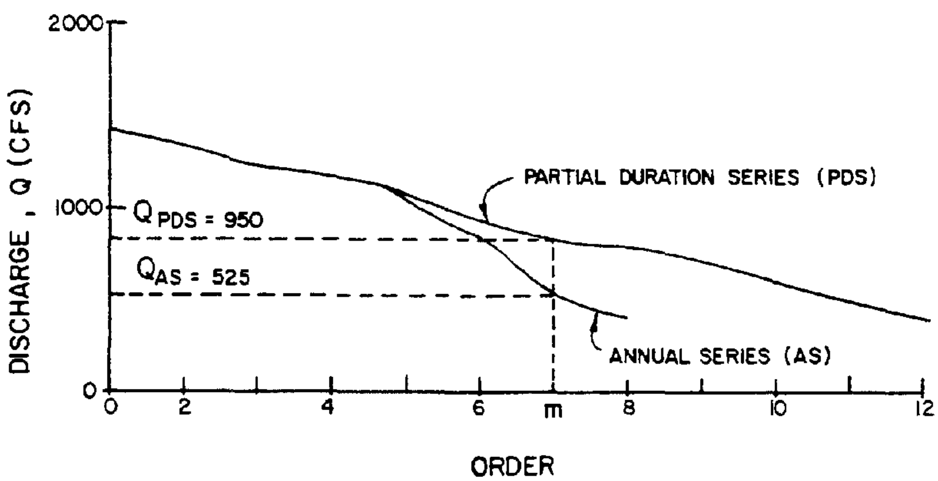



Figure 17. Annual and Partial Duration Series

higher peak flow than the Annual Series. The difference is greatest at the lower flows and becomes very small at the higher peak discharges. If the recurrence interval of these peak flows is computed as the order divided by the number of events (not years), the recurrence interval of the Partial Duration Series can be computed in the terms of the Annual Series by the equation

$$T_B = \frac{1}{\ln T_A - \ln (T_A - 1)} \qquad (4\text{-}1)$$

where $T_B$ and $T_A$ are the recurrence intervals of the Partial Duration Series and Annual Series respectively. Equation (4-1) can also be plotted as shown in Figure 18.

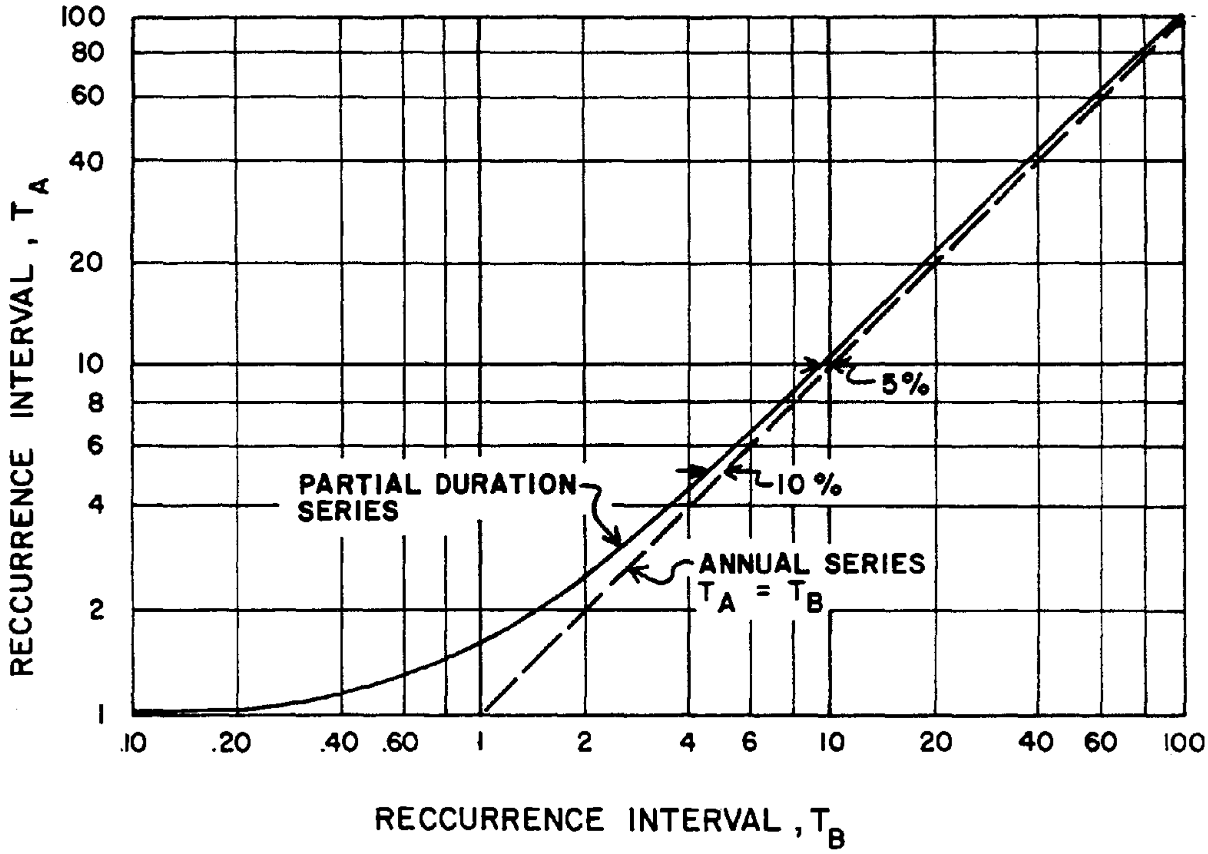



Figure 18. Relation Between Annual and Partial Duration Series

This curve shows that the maximum deviation between the two series occurs for flows with recurrence intervals less than 10 years. At this interval the deviation is about 5 percent and for the 5-year discharge, the deviation is about 10 percent. For the less frequent floods, the two series approach one another, see Table 4 below.

The Partial Duration Series is most useful for determining floods with intervals less than 10 years and for making economic analyses and subsequent risk evaluations. It is sometimes difficult to obtain data on secondary floods and it is often necessary to have stage data in order to determine the peak flows for events less severe than the peak annual flood.

Table 4. Comparison of Annual and Partial Duration Curves

| Annual-event curve (No. of years flow is exceeded per hundred years) | Partial-duration curve (No. of times flow is exceeded per hundred years) |
|---|---|
| 1.00 | 1.00 |
| 2.00 | 2.02 |
| 5.00 | 5.10 |
| 10.00 | 10.50 |
| 20.00 | 22.30 |
| 30.00 | 35.60 |
| 40.00 | 51.00 |
| 50.00 | 69.30 |
| 60.00 | 91.70 |
| 63.20 | 100.00 |
| 70.00 | 120.00 |
| 80.00 | 161.00 |
| 90.00 | 230.00 |
| 95.00 | 300.00 |

from Beard, 1962

When using the Partial Duration Series, one must be especially careful that the selected flood peaks are independent events. This is a tough practical problem since secondary flood peaks may occur during the same flood as a result of high antecedent moisture conditions. In this case, the secondary flood is not an independent event. One should also be cautious with the choice of the lower limit or base flood since it directly affects the computation of the properties of the distribution (i.e. the mean value, the variance and standard deviation and the coefficient of skew) all of which may change the peak flow determinations. For this reason it is probably best to utilize the Annual Series and convert the results to a Partial Duration Series through use of Equation (4-1). For the less frequent events, (greater than 5 to 10 years), the Annual Series is entirely appropriate and no other analysis is required.

### 4.2.2 Arrangement by Time of Occurrence

Another way to arrange data is according to its time of occurrence. Such an arrangement is called a time series. As an example of a time series the same 29 years of data presented in Table 3, are arranged according to year of occurrence rather than magnitude and plotted in Figure 19.

This time series shows the temporal variation of the data and is an important step in data analysis. The analysis of time variations is called trend analysis and there are several methods which are used in trend analysis. The two most commonly used in hydrologic analysis are the moving average method and the methods of curve fitting. The various methods of curve fitting are discussed in more detail in the reference by Sanders, 1980. The method of

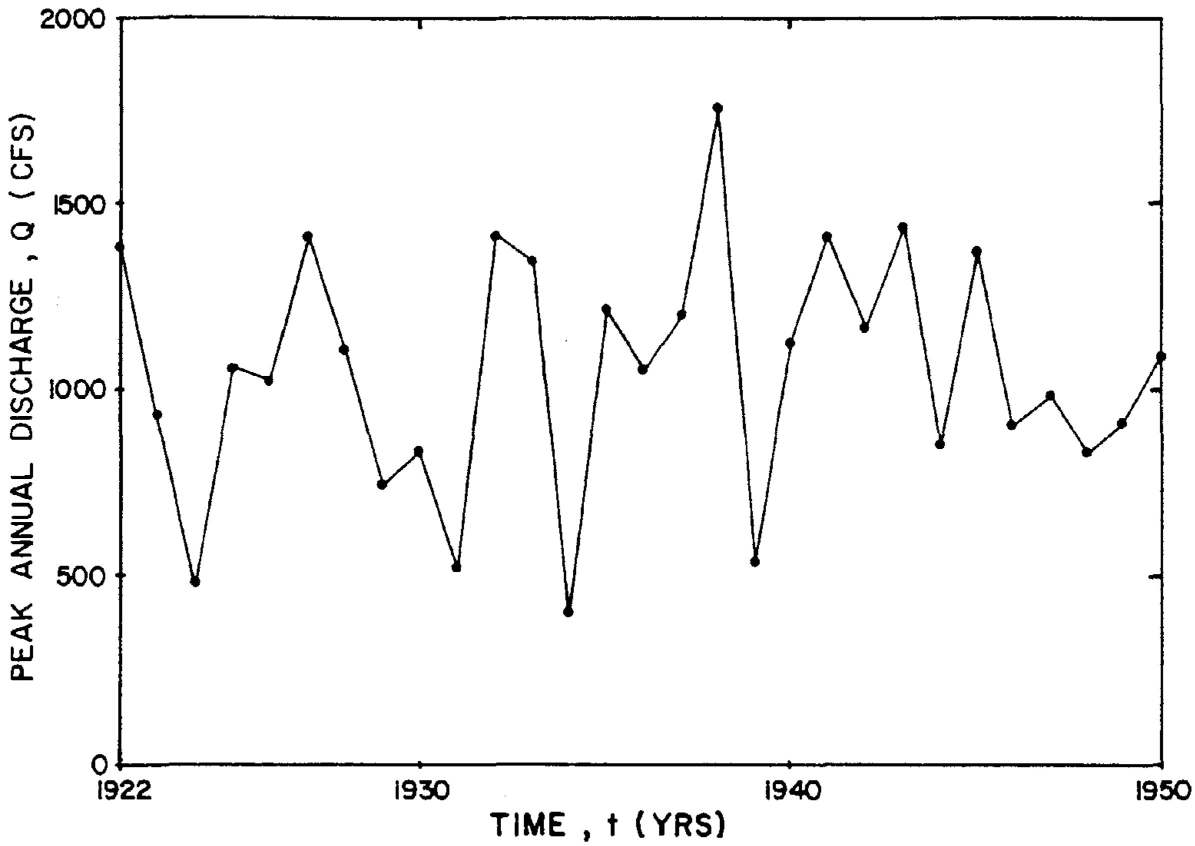



Figure 19. Time Series, Mono Creek, CA

moving averages is presented here. In the moving average method, the trend is analyzed by taking a succession of averages for a certain number of items. This succession of averages will tend to smooth out variations, and a better picture of the trend is provided. To illustrate the use of the moving average method, the 5-year moving average for the 29 years of data on Mono Creek has been computed in Table 5 and plotted in Figure 20.

Trend analysis plays an important role in evaluating the effects of changing land use and other time dependent parameters. Often through the use of trend analysis, future events can be estimated more rationally.

4.2.3 Arrangement by Geographic Location

The primary purpose of arranging flood data by geographic area is to develop a data base for the analysis of peak flows at sites that are either ungaged or have insufficient data. Classically, flood data are grouped for basins with similar meteorologic and physiographic characteristics. Meteorologically, this means that floods are caused by storms with similar type rainfall intensities, durations, distributions, shapes, travel directions, and other

Table 5. Computation of 5-Year Moving Average of Peak Flows, Mono Creek, CA

| Year | Floods Q~cfs | 5 year peak avg. | Year | Floods Q~cfs | 5 year peak avg. |
|---|---|---|---|---|---|
| 1922 | 1390 | | 37 | 1210 | 1051 |
| 23 | 940 | | 38 | 1760 | 1133 |
| 24 | 480 | | 39 | 540 | 1160 |
| 1925 | 1060 | | 1940 | 1130 | 1140 |
| 26 | 1030 | 982 | 41 | 1420 | 1212 |
| 27 | 1420 | 988 | 42 | 1170 | 1204 |
| 28 | 1110 | 1022 | 43 | 1440 | 1140 |
| 29 | 750 | 1074 | 44 | 855 | 1203 |
| 1930 | 848 | 1032 | 1945 | 1370 | 1251 |
| 31 | 525 | 931 | 46 | 910 | 1149 |
| 32 | 1420 | 931 | 47 | 988 | 1113 |
| 33 | 1350 | 979 | 48 | 838 | 992 |
| 34 | 404 | 909 | 49 | 916 | 1004 |
| 1935 | 1230 | 986 | 1950 | 1100 | 950 |
| 36 | 1060 | 1093 | | | |

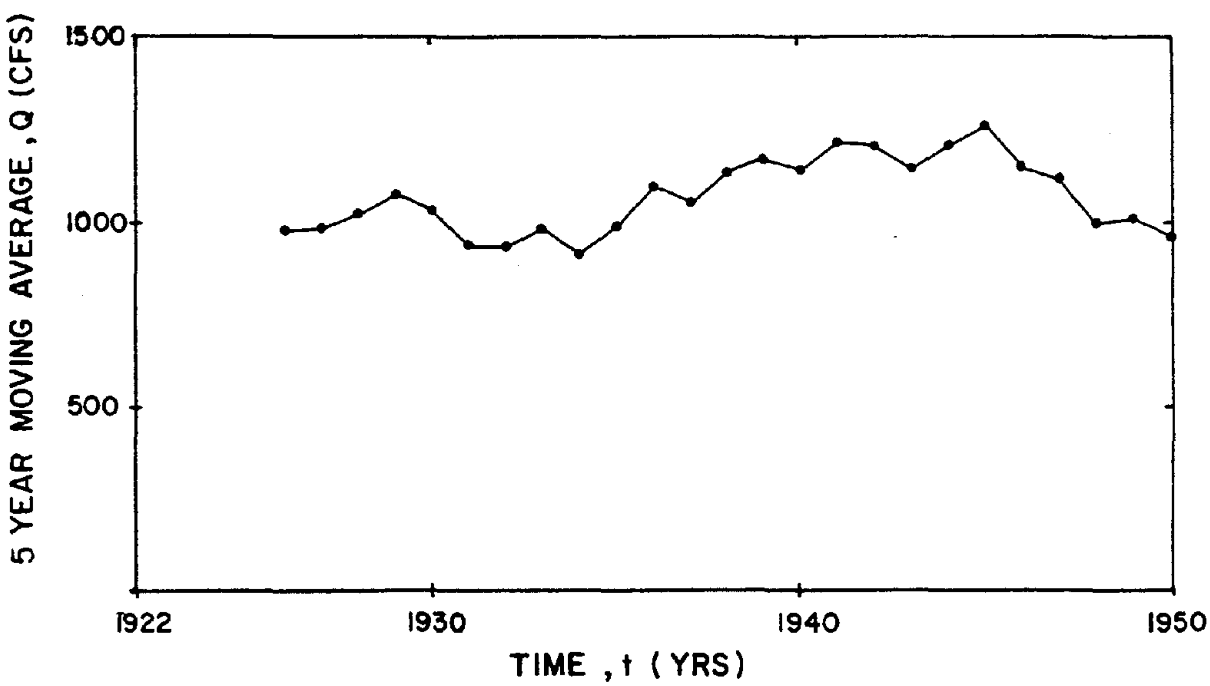



Figure 20. 5-Year Moving Average, Mono Creek, CA

climatic conditions. Similarity of physiographic features means that basin slopes, shapes, stream density, ground cover, geology and hydrologic abstractions are similar among different watersheds.

Some of these parameters are described quantitatively in a variety of ways while others are totally subjective. Therefore, there can be considerable variation in estimates of watershed similarity in a geographical area. From a quantitative standpoint, it is preferable to consider the properties which describe the distribution of floods from different watersheds. These properties, which are described more fully in later parts of this section, include variance, standard deviation and coefficient of skew. Other tests can be used to test for hydrologic homogeneity such as the runoff per unit of drainage area, the ratio of various frequency floods to average floods, the standard error of estimate and deviates from regression analyses. The latter techniques are typical of those used to establish geographic areas for regional regression equations and other regional procedures for peak flow estimates.

### 4.2.4 Probabilistic Concepts

The statistical analysis of repeated observations of an event, e.g. observations of peak annual flows, is based on the laws of probability. The probability of occurrence of a single peak flow, $Q_1$, is the relative number of occurrences of $Q_1$ after a long series of observations, i.e.

$$\Pr\{Q_1\} = \frac{n_1}{n} = \frac{\text{No. of occurrences of some flood}}{\text{No. of observations (if large)}} \tag{4-2}$$

where $n_1$ is defined as the frequency and $n_1/n$ is the relative frequency of $Q_1$.

Most people have an intuitive grasp of the concept of probability. They know that if a coin is tossed, there is an equal probability that a head or a tail will result. They know this because there are only two possible outcomes and that each is equally likely. Again, relying on past experience or intuition, when a fair die is tossed, there are six equally likely outcomes, any of the numbers 1, 2, 3, 4, 5, or 6. Each has a probability of occurrence of 1/6. So the chances that the number 3 will result from a single throw is 1 out of 6. This is fairly straightforward because all of the possible outcomes are known beforehand and the probabilities can be readily quantified.

On the other hand, the probability of a nonoccurrence (or failure) of an event such as peak flow, $Q_1$, is given by

$$\Pr\{\text{not } Q_1\} = \frac{n-n_1}{n} = 1 - \frac{n_1}{n} = 1 - \Pr\{Q_1\} \tag{4-3}$$

Combining Equations (4-2) and (4-3) it is seen that

$$\Pr\left\{Q_1\right\} + \Pr\left\{\text{not } Q_1\right\} = 1 \tag{4-4}$$

or that the probability of an event occurring is between 0 and 1, i.e. $0 \leq \Pr\{Q_1\} \leq 1$. If an event is certain to occur, its probability is 1, and if it cannot occur at all, its probability is 0.

Given two independent flows $Q_1$ and $Q_2$, the probability of the successive or simultaneous occurrence of both $Q_1$ and $Q_2$ is given by

$$\Pr\left\{Q_1 \text{ and } Q_2\right\} = \Pr\left\{Q_1\right\} \Pr\left\{Q_2\right\} \tag{4-5}$$

If the occurrence of a flow $Q_1$ excludes the occurrence of another flow $Q_2$, then the two events are said to be mutually exclusive. The probability of occurrence of either $Q_1$ or $Q_2$ is given by

$$\Pr\left\{Q_1 \text{ or } Q_2\right\} = \Pr\left\{Q_1\right\} + \Pr\left\{Q_2\right\} \tag{4-6}$$

4.2.5 Return Period

If the probability of a given annual peak flow, $Q_1$, or its relative frequency determined from Equation (4-2) is 0.2, this means there is an equal chance of 20 percent that this flood over a long period of time will be exceeded in each year. Stated another way, this flood will be exceeded on an average of once every 5 years. This is called the return period or recurrence interval.

The return period, Tr, is given by

$$Tr = \frac{1}{\Pr\left\{Q_1\right\}} \tag{4-7}$$

The designer is cautioned to remember that a flood with a return period of 5 years does not mean this flood will occur once every five years. As noted, the flood has a 20 percent probability of occurring in any year, and there is no preclusion of the 5-year flood occurring in several consecutive years. The same is true for any flood of specified return period.

### 4.2.6 Risk

The probability of nonoccurrence of $Q_1$, Equation (4-3) can now be written in terms of the return period as

$$Pr\{\text{not } Q_1\} = 1 - Pr\{Q_1\} = 1 - \frac{1}{Tr} \tag{4-8}$$

The probability that $Q_1$ will not occur for n successive years is given by Equation (4-8) as

$$Pr_1\{\text{not } Q_1\} \times Pr_2\{\text{not } Q_1\} \times \cdots \times Pr_n\{Q_1\} = Pr\{\text{not } Q_1\}^n = \left(1 - \frac{1}{Tr}\right)^n \tag{4-9}$$

Risk, R, is defined as the probability that $Q_1$ will occur at least once in n years, or

$$R = 1 - Pr\{\text{not } Q_1\}^n = 1 - \left(1 - \frac{1}{Tr}\right)^n \tag{4-10}$$

Equation (4-10) can be used to calculate Table 6 which gives the risk of failure as a function of project design life, n, and the design return period, Tr.

The use of Equation (4-10) or Table 6 is illustrated by the following example. What is the risk that a design flood will be equaled or exceeded in the first two years on a frontage road culvert designed for a 10-year flood? From Equation (4-10), the risk is calculated as

$$R = 1 - \left(1 - \frac{1}{Tr}\right)^n = 1 - \left(1 - \frac{1}{10}\right)^2 = 0.19$$

In other words, there is about a 20 percent chance this structure will be subject to the 10-year design storm in the first two years of its life.

The use of Risk Analysis and the relations cited in this section are discussed in more detail in Sec. 9.0 of this manual.

### 4.2.7 Frequency Distribution Concepts

The typical problem faced in hydrology involves situations where all possible floods (or outcomes) are unknown. In order to address the question of the probability of a certain flood from a sample of an infinite population, actual data form the basis for the statistical analysis of some future flood event.

Table 6. Risk as a Function of Project Life and Return Period

| Permissible risk of failure (R) | Project life in years (n) | | | |
|---|---|---|---|---|
| | 1 | 25 | 50 | 100 |
| | Required return period (1/P)= Tr(years) | | | |
| 0.01 | 100 | 2,440 | 5,260 | 9,100 |
| 0.25 | 4 | 87 | 175 | 345 |
| 0.50 | 2 | 37 | 72 | 145 |
| 0.75 | 1.3 | 18 | 37 | 72 |
| 0.99 | 1.01 | 6 | 11 | 27 |

To facilitate an analysis of this type, the concepts of frequency distributions are utilized. A frequency distribution is simply an arrangement of data by classes or categories with associated frequencies of each class. The frequency distribution can then be used to obtain information on the magnitude of past events, as well as how often events of a specified magnitude have occurred.

A frequency distribution is constructed by first examining the range of magnitudes, i.e. the difference between the largest and the smallest floods, and dividing this range into a number of conveniently sized groups, usually between 10 and 20. These groups are called class intervals. The size of the class interval is simply the range divided by the number of class intervals selected. There is no hard and fast rule concerning the number of class intervals to select, but the following guidelines may be helpful.

1. The class intervals should not overlap; 0-99, 100-199, etc., should be used in preference to 0-100, 100-200, etc.

2. The number of class intervals should be chosen so that there are not too many class intervals which do not have any events.

3. The class intervals should be of uniform size.

Using these rules, the discharges for Mono Creek listed in Table 3 are arranged according to class intervals as shown in Table 7 below.

This data can also be represented graphically by a Frequency Histogram as shown in Figure 21. Since relative frequency has been defined as the number of occurrences of a certain class of events divided by the period of record, this curve also represents Pr{Q} as shown on the right hand ordinate of Figure 21.

Table 7. Arrangement of Flood Data by Class Intervals, Mono Creek, CA

| Mean Annual Flow | Number of Occurrences | No. Times Equaled or Exceeded | Relative Frequency | Cumulative Frequency |
|---|---|---|---|---|
| 0- 199 | 0 | 0-29 | 0 | 0 |
| 200- 399 | 0 | 200-29 | 0 | 0 |
| 400- 599 | 4 | 400-29 | .14 | .14 |
| 600- 799 | 1 | 600-25 | .03 | .17 |
| 800- 999 | 7 | 800-24 | .24 | .41 |
| 1000-1199 | 7 | 1000-17 | .24 | .65 |
| 1200-1399 | 5 | 1200-10 | .17 | .82 |
| 1400-1599 | 4 | 1400-5 | .14 | .96 |
| 1600-1799 | 1 | 1600-1 | .03 | .99 |

From this Frequency Histogram, several features of the data can now be illustrated. Notice that there are some magnitudes which have occurred more frequently than others; also notice that the data is somewhat spread out and that it is not symmetrical. These are features of every frequency distribution and they have special names and means of measurement.

4.2.7.1 Central Tendency

The clustering of the data about particular magnitudes is known as central tendency of which there are a number of measures. The most frequently used is the average or the mean value. The mean value is calculated by summing all of the individual values of the data and dividing the total by the number of individual data values as shown by Equation (4-11)

$$\bar{Q} = \frac{\sum_{i=1}^{n} Q_i}{n} \qquad (4\text{-}11)$$

The symbol $\bar{Q}$ is used for an average or mean flow. The symbol, $\Sigma$, means the summation of all flow values between the two indicated values of the indices (1 and n in the case above). Another measure of central tendency used is the median. The median is the value of the middle item when the items are arranged according to magnitude. When there are an even number of items, the median is taken as the average of the values of the two central items. Still another measure of central tendency which is sometimes used is the mode. The mode is the most frequent or most common value which occurs in a set of data.

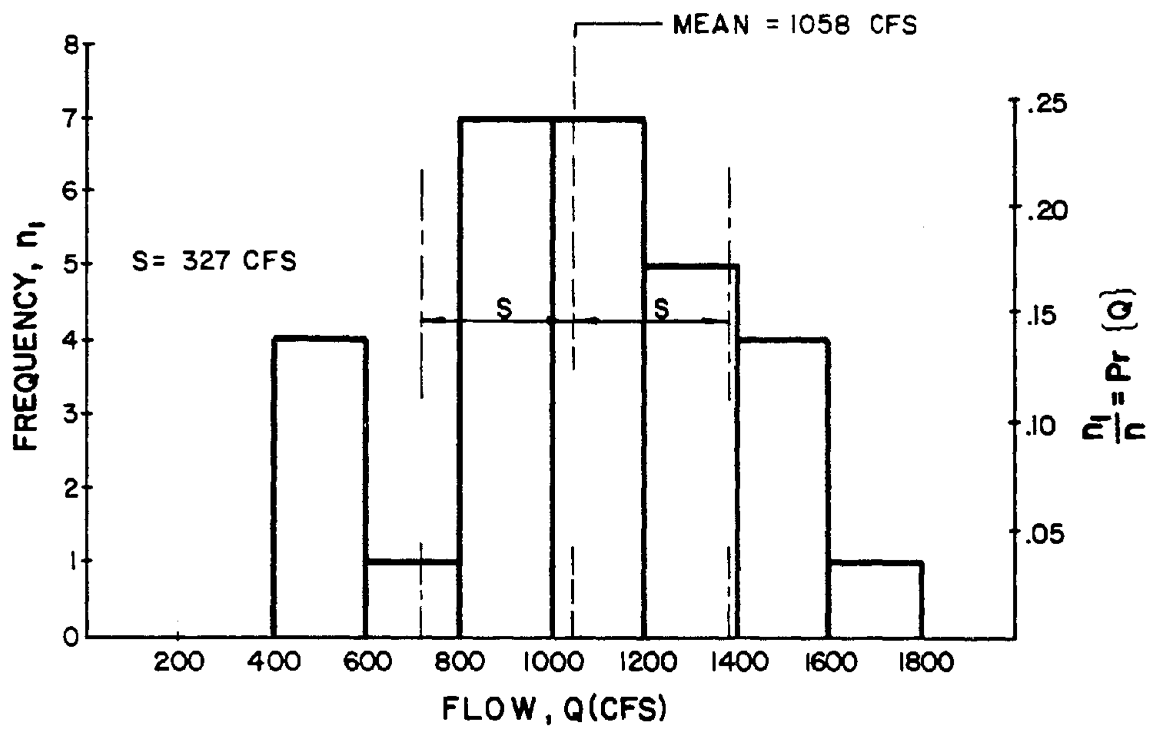



Figure 21. Flood Frequency Histogram, Mono Creek, CA

4.2.7.2 Variability

The spread of the data is called dispersion and it also has measures. The most commonly used measure of dispersion is the standard deviation. The standard deviation, S, is defined as the square root of the mean square of the deviations from the average value. This is shown symbolically as

$$S = \sqrt{\frac{\sum_{i=1}^{n} (Q_i - \bar{Q})^2}{n-1}} = \bar{Q} \sqrt{\frac{\sum_{i=1}^{n} \left(\frac{Q_i}{\bar{Q}} - 1\right)^2}{n-1}} \qquad (4\text{-}12)$$

The second expression on the right hand side of Equation (4-12) is often used to facilitate and improve on the accuracy of hand calculations.

Another measure of dispersion of the flood data is the variance, or simply the standard deviation squared. A measure of relative dispersion is the coefficient of variance, V, or the standard deviation divided by the mean flow.

$$V = \frac{S}{\bar{Q}} \qquad (4\text{-}13)$$

#### 4.2.7.3 Skewness

The symmetry of the frequency distribution, or more accurately the asymmetry, is called skewness. The measure of skew is the coefficient of skewness, G. The skew coefficient is calculated by Equation (4-14)

$$G = \frac{n \sum_{i=1}^{n} (Q_i - \bar{Q})^3}{(n-1)(n-2) S^3} = \frac{\sum_{i=1}^{n} \left( \frac{Q_i}{\bar{Q}} - 1 \right)^3}{(n-1) V^3} \tag{4-14}$$

where all symbols are as previously defined. Again, the second expression on the right hand side of the equation is for ease of hand computations.

If the frequency distribution were perfectly symmetrical, the coefficient of skew would be zero. If the distribution were to have a longer "tail" to the right of the central maximum than to the left, the distribution would have a positive skewness and G would be positive. If the longer tail were to the left of the central maximum, then the distribution would have a negative coefficient of skew.

Table 8 illustrates the computation of measures of central tendency, standard deviation, variance and coefficient of skew for the Mono Creek Frequency Distribution shown in Figure 21. Computed values of the mean and standard deviation are also identified in Figure 21.

Table 8 shows that the mean value of the sample of floods is 1057.6 CFS (30.0 CMS), the standard deviation is 327.3 CFS (9.3 CMS) and the coefficient of variance is 0.309. The coefficient of skew is -0.151 meaning the distribution is skewed negatively to the left. For the flow data in Table 8, the median value is 1060 CFS (30 CMS) and the most frequent value, or mode is 1420 CFS (40 CMS).

The three main characteristics of the frequency distribution, mean, standard deviation and coefficient of skew are very important parameters and will be used many times in subsequent sections of this manual.

### 4.2.8 Probability Distribution Functions

If the frequency distribution histogram from a very large population of floods were constructed, it would be possible to define very small class intervals and still have a number of events in each interval. Under these conditions the frequency distribution histogram would approach a smooth curve as shown in Figure 22.

Table 8. Computation of Statistical Characteristics of Mono Creek, CA

| Year | Flood Q, cfs | Floods in order | Order | $\frac{Q_i}{\bar{Q}}$ | $\frac{Q_i}{\bar{Q}} - 1$ | $(\frac{Q_i}{\bar{Q}} - 1)^2$ | $(\frac{Q_i}{\bar{Q}} - 1)^3$ |
|---|---|---|---|---|---|---|---|
| 1922 | 1390 | 1760 | 1 | 1.669 | 0.669 | 0.447 | 0.2990 |
| 23 | 940 | 1440 | 2 | 1.362 | 0.362 | 0.131 | 0.0475 |
| 24 | 488 | 1420 | 3 | 1.345 | 0.345 | 0.119 | 0.0410 |
| 1925 | 1060 | 1420 | 4 | 1.345 | 0.345 | 0.119 | 0.0410 |
| 26 | 1030 | 1420 | 5 | 1.345 | 0.345 | 0.119 | 0.0410 |
| 27 | 1420 | 1390 | 6 | 1.318 | 0.318 | 0.101 | 0.0321 |
| 28 | 1110 | 1370 | 7 | 1.299 | 0.299 | 0.0895 | 0.0268 |
| 29 | 750 | 1350 | 8 | 1.279 | 0.279 | 0.0778 | 0.0217 |
| 1930 | 848 | 1230 | 9 | 1.165 | 0.165 | 0.0272 | 0.0045 |
| 31 | 525 | 1210 | 10 | 1.148 | 0.148 | 0.0219 | 0.0032 |
| 32 | 1420 | 1170 | 11 | 1.109 | 0.109 | 0.0119 | 0.0013 |
| 33 | 1350 | 1130 | 12 | 1.070 | 0.070 | 0.0049 | 0.0003 |
| 34 | 404 | 1110 | 13 | 1.051 | 0.051 | 0.0026 | 0.0001 |
| 1935 | 1230 | 1100 | 14 | 1.041 | 0.041 | 0.0017 | 0.0001 |
| 36 | 1060 | 1060 | 15 | 1.003 | 0.003 | ~ 0 | ~ 0 |
| 37 | 1210 | 1060 | 16 | 1.003 | 0.003 | ~ 0 | ~ 0 |
| 38 | 1760 | 1030 | 17 | 0.975 | -0.025 | 0.0006 | ~ 0 |
| 39 | 540 | 988 | 18 | 0.935 | -0.065 | 0.0042 | -0.0003 |
| 1940 | 1130 | 940 | 19 | 0.890 | -0.110 | 0.0121 | -0.0013 |
| 41 | 1420 | 916 | 20 | 0.869 | -0.131 | 0.0172 | -0.0023 |
| 42 | 1170 | 910 | 21 | 0.861 | -0.139 | 0.0193 | -0.0027 |
| 43 | 1440 | 855 | 22 | 0.810 | -0.190 | 0.0361 | -0.0069 |
| 44 | 855 | 848 | 23 | 0.804 | -0.196 | 0.0384 | -0.0075 |
| 1945 | 1370 | 838 | 24 | 0.794 | -0.206 | 0.0425 | -0.0088 |
| 46 | 910 | 750 | 25 | 0.710 | -0.290 | 0.0841 | -0.0244 |
| 47 | 988 | 540 | 26 | 0.511 | -0.489 | 0.2390 | -0.1170 |
| 48 | 838 | 525 | 27 | 0.497 | -9.503 | 0.2530 | -0.1272 |
| 49 | 916 | 488 | 28 | 0.462 | -0.538 | 0.2900 | -0.1560 |
| 1950 | 1100 | 404 | 29 | 0.382 | -0.618 | 0.3720 | -0.2300 |
| TOTALS | 30,672 | | | | | 2.682 | -0.1248 |

$$\bar{Q} = \frac{\sum_{i=1}^{n} Q_i}{n} = \frac{30{,}672}{29} = 1057.6 \text{ CFS } (30.0 \text{ CMS})$$

$$S = \bar{Q}\sqrt{\frac{\sum_{i=1}^{n}\left(\frac{Q_i}{\bar{Q}} - 1\right)^2}{n-1}} = 1057.6\sqrt{\frac{2.6827}{28}} = 327.3 \text{ CFS } (9.3 \text{ CMS})$$

$$V = \frac{S}{\bar{Q}} = \frac{327.3}{1057.6} = 0.309$$

$$G = \frac{\sum_{i=1}^{n}\left(\frac{Q_i}{\bar{Q}} - 1\right)^3}{(n-1)V^3} = \frac{-0.1248}{(29-1)(0.309)^3} = -0.151$$

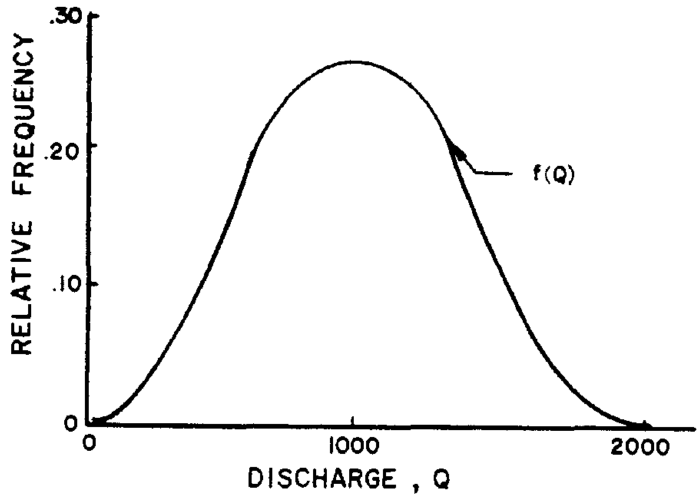



Figure 22. Probability Density Function

This curve is called the Probability Density Function, f(Q), and is defined such that

$$\int_{-\infty}^{\infty} f(Q)\,dQ = 1 \qquad (4\text{-}15)$$

This equation is a mathematical statement that the sum of the probabilities of all events is equal to unity. From Equation (4-15), two conditions of hydrologic probability are readily illustrated from the Probability Density Function. Figure 23a shows that the probability of a flow Q falling between two known flows, $Q_1$ and $Q_2$, is the area under the Probability Density Curve between $Q_1$ and $Q_2$.

Figure 23b shows the probability that a flood Q exceeds $Q_1$ is the area under the curve from $Q_1$ to infinity.

As can be seen from Figures 23a and 23b, the calculation for probability from the frequency distribution function is somewhat tedious. A further refinement of the frequency distribution is the Cumulative Frequency Distribution. The flood data presented in Table 7 can be used to illustrate the development of a cumulative frequency distribution which is simply the cumulative total of the relative frequencies by class interval. For each range of flows, Column 3 of the Table defines the number of times floods equal or exceed the lower limit of the class interval and Column 5 gives the cumulative frequency. Using the

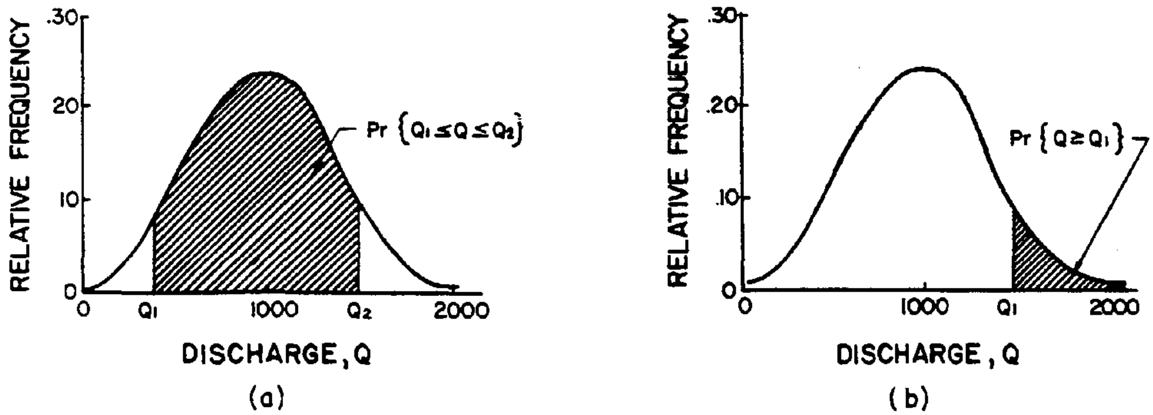



Figure 23. Hydrologic Probability from Density Functions

cumulative frequency distribution it is possible to compute directly the nonexceedance probability for a given magnitude. The nonexceedence probability is defined as the probability that the specified value will not be exceeded. This is an often used probability in hydrologic data analysis. The Cumulative Frequency Histogram for the Mono Creek, CA data is shown in Figure 24.

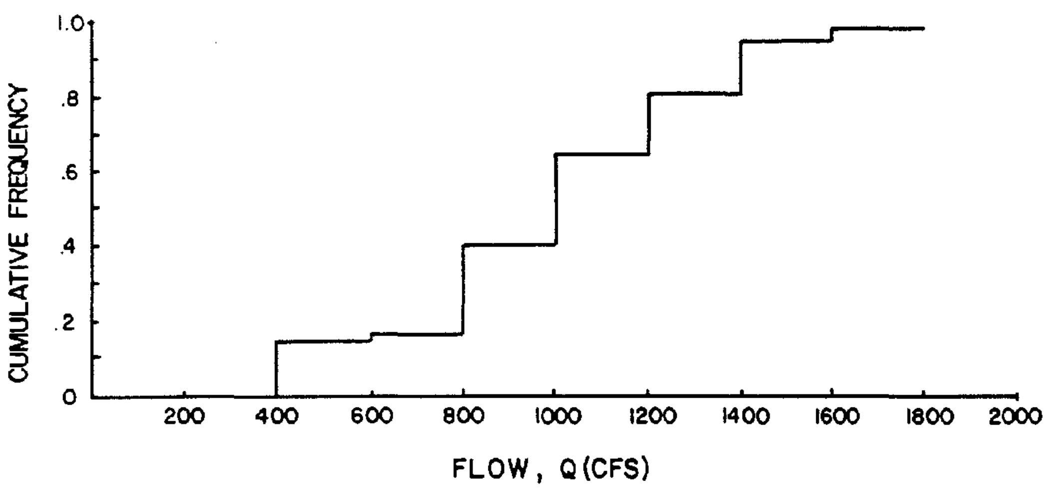



Figure 24. Cumulative Frequency Histogram, Mono Creek, CA

Again, if the sample were very large so that small class intervals could be defined, the histogram becomes a smooth curve which is defined as the Cumulative Probability Functions, F(Q), and shown in Figure 25a. This figure is actually a plot of the area under the curve (the sum of the probabilities) of Figure 22 and defines the probability that the flow will be less than some stated value.

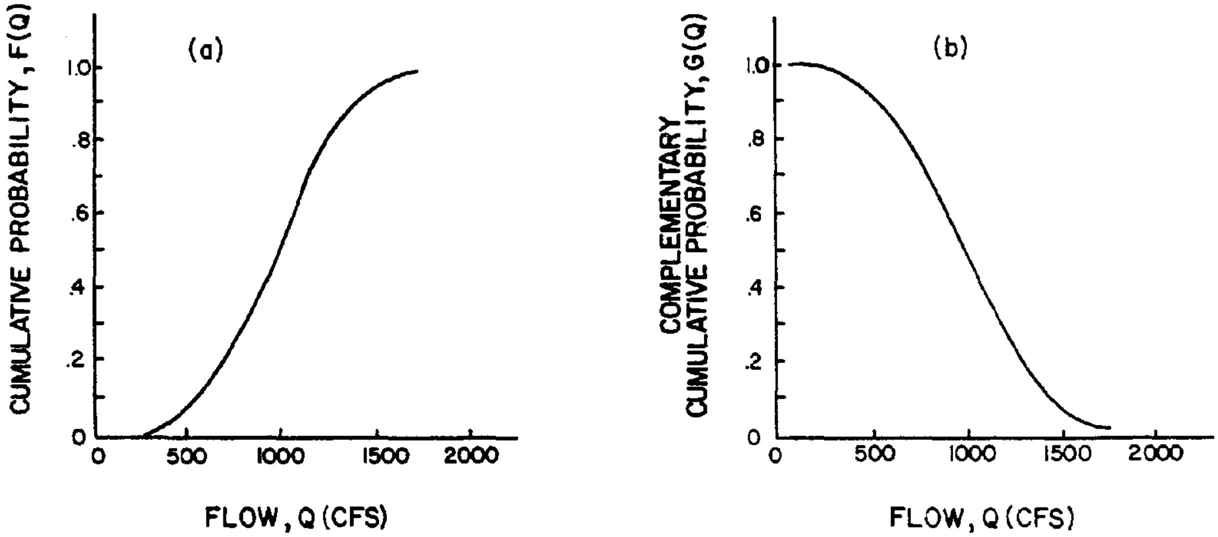



Figure 25. Cumulative and Complementary Cumulative Functions

Another convenient representation for hydrologic analysis is the Complementary Probability Function, G(Q), defined as

$$G(Q) = 1 - F(Q) = Pr\left\{Q \geq Q_1\right\} \tag{4-16}$$

The function, G(Q), shown in Figure 25b is the exceedance probability, i.e. the number of times a flow of a given magnitude is equaled or exceeded.

## 4.3 Standard Frequency Distributions

Several frequency distributions keep recurring in the analysis of hydrologic data, and as a result they have been studied extensively and are now standardized. The standard frequency distributions which have been found most useful in hydrologic data analysis are:

1. the normal distribution

2. the log-normal distribution

3. the Gumbel extreme value distribution

4. the log-Pearson Type III distribution

The characteristics and application of each of these distributions will be presented in the following sections.

4.3.1 Plotting Position

The application of standard frequency distributions is dependent on the probability position assigned to each flow. This probability position is commonly called the plotting position; and as will be seen in the following discussions, it defines where on the probability scale of probability graph paper, a given flow is plotted.

One such plotting position has already been defined by Equation (4-2) as the relative frequency. There are, however, a number of different formulas that have been proposed for plotting position and there is no unanimity on the preferred method. Beard, 1962, illustrates the nature of this problem. If a very long period of record, say 2000 years, is broken up into 100 20-year records and each is analyzed separately, then the highest flood in each of these 20-year records will have the same probability of occurrence of 0.05. Actually, one of these 100 highest floods is the 1 in 2000 year flood or a flood with a probability of occurrence of 0.0005. Some of the records will also contain 100-year floods and many will contain floods in excess of the 20-year flood. Similarly some of the 20-year records will contain highest floods that are less than the actual 20-year flood. Thus, the problem is to select a plotting position so that the general trend of the data will agree reasonably well with the selected frequency distribution.

Variation in plotting position formulas results from adjusting the probabilities of the various floods in the sample to its central tendency characteristics. For example, the probabilities can be adjusted to the median flow by the formula, (Beard, 1962)

$$P = 1-(0.5)^{1/n} \tag{4-17}$$

where P is the plotting position for the largest event and n is the number of years of record. The plotting position for the smallest flood is the complement of Equation (4-17) and all intermediate values are linearly interpolated. Equation (4-17) will tend to give probabilities that are too high for half the data and too low for the other half.

Plotting position, P, can also be corrected to the mean flow by the Weibull Formula

$$P = \frac{m}{n+1} \tag{4-18}$$

where m is the rank and n is the number of years of record. Equation (4-18) is one of the more commonly accepted formulas and will be used in subsequent discussions and examples. For the Partial-Duration Series where the number of floods exceeds the number of years of record, Beard, 1962, recommends

$$P = \frac{2m - 1}{2n} \tag{4-19}$$

where m is the order number of the event.

### 4.3.2 Normal Distribution

The normal or Gaussian distribution is a classical mathematical distribution occurring in the analysis of natural phenomena. The normal distribution is a symmetrical, unbounded, bell-shaped curve with the maximum value at the central point and extending from $-\infty$ to $+\infty$. A typical normal distribution is shown in Figure 26.

For the normal distribution, the maximum central value occurs at the mean flow. Because of absolute symmetry, half of the flows are below the mean and half are above. Therefore, the median corresponds to the mean value. Another characteristic of the normal distribution curve is that 68.3 percent of the events will fall between $\pm$ one standard deviation, 95 percent of the events will fall within $\pm 2S$, and 99.7 percent will fall within $\pm 3S$.

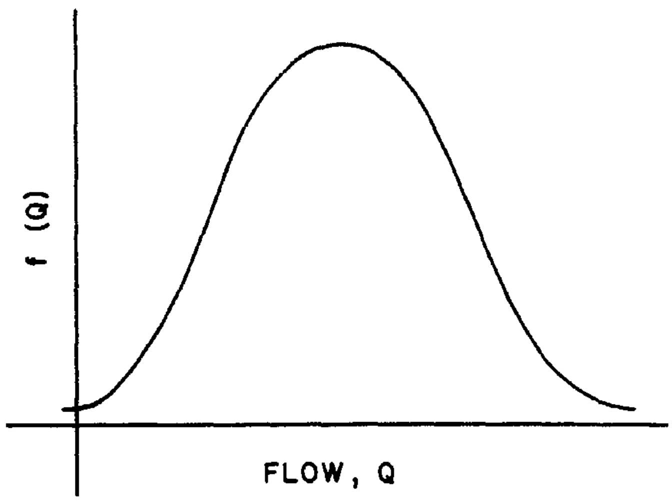



Figure 26. Normal Distribution Curve

The coefficient of skew is zero. The function describing the normal distribution curve is

$$f(Q) = \frac{1}{S\sqrt{2\pi}} e^{-\frac{(Q-\bar{Q})^2}{2S^2}} \qquad (4\text{-}20)$$

Note that only two parameters are necessary to describe the normal distribution -- the mean value, $\bar{Q}$, and the standard deviation, S.

As noted in Section 4.2.8, the cumulative frequency distribution, or the integral of Equation (4-20) is more convenient for hydrologic analysis since it permits the exceedance frequency to be related directly to flow. Values of the cumulative distribution function or the integral of Equation (4-20) are tabulated in abbreviated form for selected exceedance probabilities in Table 9 for the normal distribution at zero skew.

In order to further facilitate the analysis of data, special arithmetic-probability paper, available commercially, has been developed which has a specially transformed horizontal probability scale. The horizontal scale is transformed in such a way that the cumulative distribution function for a normal distribution will plot as as straight line. If a series of peak flows that are normally distributed are plotted against the cumulative frequency function or the exceedance frequency on the probability scale, the data will plot as a straight line with the equation

$$Q = \bar{Q} + KS \qquad (4\text{-}21)$$

where Q is the flood flow at a specified frequency and K is the value in Equation (4-20) taken from Table 9.

Table 9. Cumulative Distribution Function for Normal Distribution

| Coef. of Skew | Exceedance Probability in % | | | | | | |
|---|---|---|---|---|---|---|---|
| | 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
| | Corresponding Return Period in Years | | | | | | |
| | 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 0.0 | 0.0000 | 0.8416 | 1.2816 | 1.7507 | 2.0538 | 2.3264 | 2.8782 |

To illustrate the use of Equation (4-21) and probability paper, consider the data of Table 10. These data are the annual peak floods for the Medina River near San Antonio, Texas for the period 1940-1982 (43 years of record). Table 10 shows the calculations of the mean flow, standard deviation and coefficient of skew for these data in acccordance with Equations (4-11), (4-12), (4-13) and (4-14). Assuming the data are normally distributed, the 10- and 100-year floods are computed from Equation (4-21) as shown in Figure 27. The 10-year flood is 15,672 CFS (443.8 CMS) and the 100-year flood is 23,058 CFS (653.0 CMS). When plotted on arithmetic probability paper, these two points are sufficient to establish the straight line on Figure 27 represented by Equation (4-21).

Also plotted in Figure 27 are the actual data. The correspondence between the normal frequency curve and the actual data is poor. Obviously, the data are not normally distributed. This, however, was known beforehand (Table 10) where the data was found to have a definite right skew ($G = 2.273$).

Another disadvantage of the normal distribution is that it is unbounded in the negative direction whereas most hydrologic variables are bounded and can never be less than zero. For this reason and the fact that many hydrologic variables exhibit a pronounced skew, the normal distribution usually has limited applications. However, these problems can sometimes be overcome by performing a log transform on the data. Often the logarithms of hydrologic variables are normally distributed.

### 4.3.3 Log-Normal Distribution

The log-normal distribution has the same characteristics as the normal distribution except that the independent variable, Q, is replaced with its logarithm. The characteristics of the log-normal distribution are that it is bounded on the left by zero and it has a pronounced positive skew. These are both characteristics of many of the frequency distributions which result from an analysis of hydrologic data.

If a logarithmic transformation is performed on the normal distribution function, Equation (4-20), the resulting logarithmic distribution is often normally distributed. This enables the K values tabulated in Table 9 for a normal distribution to be used in a log-normal frequency analysis when $G_L$, the skew coefficient of the log-transformed flows, is zero. For skewed logarithmic distributions, Table 11 can be used to obtain appropriate K values.

As was the case with the simple normal distribution, a standard log-normal probability paper has been developed, where the plot of the cumulative distribution function is a straight line. This paper which is also available commercially, has a transformed horizontal scale based upon the probability function of the normal distribution and a logarithmic vertical scale. If the logarithms of the peak flows are normally distributed, the data will plot as a straight line on log-probability graph paper according to the equation

Table 10. Example Computations for Standard Normal Frequency Distribution Medina River, Texas

Basin: Medina River at San Antonio, Texas (Gage 08181500)

Location: Latitude 29° 15' 14", Longitude 98° 28' 20" - left bank of downstream side of pier of upstream bridges on U.S. 281, 6.8 miles upstream from mouth and 7 miles south of San Antonio.

Drainage Area: 1,317 square miles

Remarks: Records good. Flow slightly regulated 60 miles upstream

Record: October 1929 to December 1930, July 1939 to current year.

| Water Year | Flood Q, cfs | Floods in Order | Order | $\frac{m}{n+1}$ Plotting Position | $\frac{Qi}{\bar{Q}}$ | $(\frac{Qi}{\bar{Q}} - 1)$ | $(\frac{Qi}{\bar{Q}} - 1)^2$ | $(\frac{Qi}{\bar{Q}} - 1)^3$ |
|---|---|---|---|---|---|---|---|---|
| 1940 | 2540 | 31900 | 1 | .0227 | 4.8315 | 3.8315 | 14.6806 | 56.2490 |
| 41 | 6890 | 31800 | 2 | .0454 | 4.8163 | 3.8163 | 14.5647 | 55.5846 |
| 42 | 17500 | 17500 | 3 | .0681 | 2.6505 | 1.6505 | 2.7242 | 4.49640 |
| 43 | 12100 | 17400 | 4 | .0909 | 2.6353 | 1.6353 | 2.6744 | 4.37375 |
| 44 | 2000 | 14500 | 5 | .1136 | 2.1961 | 1.1961 | 1.4307 | 1.71141 |
| 45 | 3540 | 13100 | 6 | .1363 | 1.9841 | .9841 | .9684 | .953072 |
| 46 | 31800 | 12100 | 7 | .1590 | 1.8326 | .8326 | .6933 | .577276 |
| 47 | 1470 | 9680 | 8 | .1818 | 1.4661 | .4661 | .2172 | .101272 |
| 48 | 2050 | 9440 | 9 | .2045 | 1.4297 | .4297 | .1847 | .079378 |
| 49 | 17400 | 9220 | 10 | .2272 | 1.3964 | .3964 | .1571 | .062310 |
| 1950 | 5660 | 8160 | 11 | 0.25 | 1.2359 | .2359 | .0556 | .013128 |
| 51 | 2150 | 7510 | 12 | .2727 | 1.1374 | .1374 | .0188 | .002597 |
| 52 | 801 | 6890 | 13 | .2954 | 1.0435 | .0435 | .001897 | .000083 |
| 53 | 4960 | 6360 | 14 | .3181 | .9632 | -.0367 | .001349 | -.000050 |
| 54 | 865 | 5660 | 15 | .3409 | .8572 | -.1427 | .020376 | -.002907 |
| 55 | 1200 | 5480 | 16 | .3636 | .8299 | -.1700 | .028903 | -.004913 |
| 56 | 1750 | 5430 | 17 | .3863 | .8224 | -.1775 | .031535 | -.005600 |
| 57 | 5180 | 5180 | 18 | .4090 | .7845 | -.2154 | .046417 | -.010000 |
| 58 | 9220 | 4960 | 19 | .4318 | .7512 | -.2487 | .061885 | -.015395 |
| 59 | 3350 | 4750 | 20 | .4545 | .7194 | -.2805 | .078721 | -.022087 |
| 1960 | 3200 | 4620 | 21 | .4772 | .6997 | -.3002 | .090157 | -.027071 |
| 61 | 3050 | 4130 | 22 | 0.50 | .6255 | -.3744 | .140233 | -.052514 |
| 62 | 3960 | 3960 | 23 | .5227 | .5997 | -.4002 | .160180 | -.064108 |
| 63 | 890 | 3540 | 24 | .5454 | .5361 | -.4638 | .215145 | -.099792 |
| 64 | 2140 | 3360 | 25 | .5681 | .5089 | -.4910 | .241179 | -.118443 |
| 65 | 5430 | 3350 | 26 | .5909 | .5073 | -.4926 | .242669 | -.119542 |
| 66 | 2160 | 3200 | 27 | .6136 | .4846 | -.5153 | .265568 | -.136856 |
| 67 | 5480 | 3050 | 28 | .6363 | .4619 | -.5380 | .289500 | -.155766 |
| 68 | 13100 | 2950 | 29 | .6590 | .4468 | -.5531 | .306027 | -.169294 |
| 69 | 2730 | 2739 | 30 | .6818 | .4134 | -.5865 | .344004 | -.201765 |

Table 10. Example Computations for Standard Normal Frequency Distribution Medina River, Texas (Continued)

| Water Year | Flood Q, cfs | Floods in Order | Order | $\frac{m}{n+1}$ Plotting Position | $\frac{Qi}{\bar{Q}}$ | $(\frac{Qi}{\bar{Q}} - 1)$ | $(\frac{Qi}{\bar{Q}} - 1)^2$ | $(\frac{Qi}{\bar{Q}} - 1)^3$ |
|---|---|---|---|---|---|---|---|---|
| 1970 | 3360 | 2540 | 31 | .7045 | .3847 | -.6152 | .378539 | -.232944 |
| 71 | 2950 | 2160 | 32 | .7272 | .3271 | -.6728 | .452727 | -.304617 |
| 72 | 6360 | 2150 | 33 | 0.75 | .3256 | -.6743 | .454767 | -.306679 |
| 73 | 31900 | 2140 | 34 | .7727 | .3241 | -.6758 | .456812 | -.308750 |
| 74 | 9680 | 2050 | 35 | .7954 | .3104 | -.6895 | .475424 | -.327809 |
| 75 | 4130 | 2000 | 36 | .8181 | .3029 | -.6970 | .485925 | -.338730 |
| 76 | 7510 | 1980 | 37 | .8409 | .2998 | -.7001 | .490157 | -.343165 |
| 77 | 4620 | 1750 | 38 | .8636 | .2650 | -.7349 | .540148 | -.396981 |
| 78 | 9440 | 1470 | 39 | .8863 | .2226 | -.7773 | .604282 | -.469743 |
| 79 | 4750 | 1200 | 40 | .9090 | .1817 | -.8182 | .669533 | -.547845 |
| 1980 | 1980 | 890 | 41 | .9318 | .1347 | -.8652 | .748574 | -.647668 |
| 81 | 14500 | 865 | 42 | .9545 | .1310 | -.8689 | .755140 | -.656207 |
| 82 | 8160 | 801 | 43 | .9772 | .1213 | -.8786 | .772082 | -.678414 |
| TOTALS | | 283906 | | | | | 48.2196 | 117.4386 |

$$\bar{Q} = \frac{\sum_{i=1}^{n} Q_i}{n} = 6602.5 \text{ CFS.} = (187.0 \text{ CMS})$$

$$S = \bar{Q}\sqrt{\frac{\sum_{i=1}^{n}\left(\frac{Q_i}{\bar{Q}} - 1\right)^2}{n-1}} = 6602.5 \sqrt{\frac{48.2196}{42}} = 7074.5 \text{ CFS. } (200.3 \text{ CMS})$$

$$V = \frac{S}{\bar{Q}} = \frac{7074.5}{6602.47} = 1.07$$

$$G = \frac{\sum_{i=1}^{n}\left(\frac{Q_i}{\bar{Q}} - 1\right)^3}{(n-1)\ V^3} = \frac{117.4386}{42(1.0715)^3} = 2.273$$

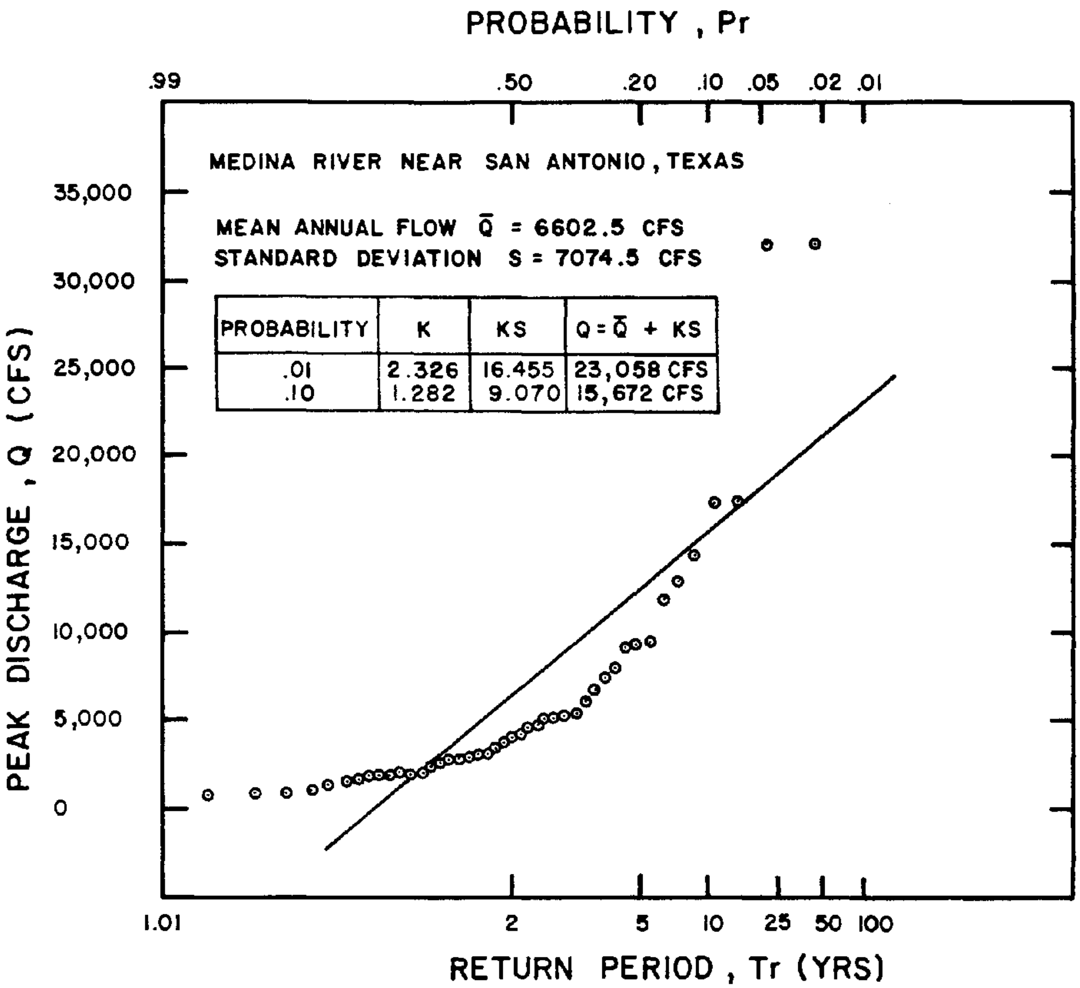



Figure 27. Normal Frequency Distribution Analysis, Medina River, Texas

$$\log Q = \bar{Q}_L + KS_L \tag{4-22}$$

where $\bar{Q}_L$ is the average of the logarithms of Q and $S_L$ is the standard deviation of the logarithmic distribution. Table 12 illustrates the computation of these values for the data series originally presented in Table 10. After converting the flows to their corresponding logarithms, the mean is 3.639, the standard deviation is 0.394 and the skew coefficient is 0.236.

Table 11. Cumulative Distribution Function for Log-Normal Distribution

| Coef. of Skew | Exceedance Probability in % | | | | | | |
|---|---|---|---|---|---|---|---|
| | 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
| | Corresponding Return Period in Years | | | | | | |
| | 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 0.1 | -0.0165 | 0.8364 | 1.2916 | 1.7847 | 2.1070 | 2.3998 | 2.9913 |
| 0.2 | -0.0334 | 0.8296 | 1.3003 | 1.8182 | 2.1611 | 2.4757 | 3.1201 |
| 0.3 | -0.0496 | 0.8220 | 1.3074 | 1.8495 | 2.2130 | 2.5497 | 3.2486 |
| 0.4 | -0.0671 | 0.8124 | 1.3134 | 1.8820 | 2.2686 | 2.6304 | 3.3924 |
| 0.5 | -0.0813 | 0.8034 | 1.3170 | 1.9078 | 2.3139 | 2.6973 | 3.5146 |
| 0.6 | -0.0957 | 0.7934 | 1.3194 | 1.9329 | 2.3597 | 2.7660 | 3.6431 |
| 0.7 | -0.1106 | 0.7817 | 1.3205 | 1.9580 | 2.4069 | 2.8383 | 3.7818 |
| 0.8 | -0.1246 | 0.7697 | 1.3201 | 1.9806 | 2.4512 | 2.9074 | 3.9179 |
| 0.9 | -0.1373 | 0.7577 | 1.3185 | 2.0002 | 2.4912 | 2.9710 | 4.0465 |
| 1.0 | -0.1503 | 0.7443 | 1.3155 | 2.0193 | 2.5320 | 3.0375 | 4.1844 |
| 1.1 | -0.1608 | 0.7326 | 1.3119 | 2.0340 | 2.5650 | 3.0923 | 4.3012 |
| 1.2 | -0.1720 | 0.7191 | 1.3069 | 2.0487 | 2.6000 | 3.1517 | 4.4309 |
| 1.3 | -0.1820 | 0.7062 | 1.3013 | 2.0610 | 2.6310 | 3.2056 | 4.5518 |
| 1.4 | -0.1911 | 0.6934 | 1.2951 | 2.0714 | 2.6593 | 3.2560 | 4.6679 |
| 1.5 | -0.1996 | 0.6809 | 1.2883 | 2.0802 | 2.6851 | 3.3030 | 4.7790 |
| 1.6 | -0.2082 | 0.6673 | 1.2803 | 2.0883 | 2.7113 | 3.3520 | 4.8979 |
| 1.7 | -0.2160 | 0.6540 | 1.2719 | 2.0949 | 2.7348 | 3.3975 | 5.0114 |
| 1.8 | -0.2235 | 0.6406 | 1.2628 | 2.1001 | 2.7568 | 3.4412 | 5.1236 |
| 1.9 | -0.2302 | 0.6276 | 1.2536 | 2.1040 | 2.7765 | 3.4815 | 5.2304 |
| 2.0 | -0.2366 | 0.6146 | 1.2438 | 2.1069 | 2.7947 | 3.5202 | 5.3357 |
| 2.1 | -0.2421 | 0.6026 | 1.2344 | 2.1085 | 2.8102 | 3.5544 | 5.4314 |
| 2.2 | -0.2471 | 0.5911 | 1.2250 | 2.1092 | 2.8239 | 3.5858 | 5.5219 |
| 2.3 | -0.2520 | 0.5792 | 1.2149 | 2.1092 | 2.8371 | 3.6171 | 5.6147 |
| 2.4 | -0.2566 | 0.5673 | 1.2045 | 2.1083 | 2.8490 | 3.6469 | 5.7059 |
| 2.5 | -0.2605 | 0.5565 | 1.1947 | 2.1068 | 2.8589 | 3.6730 | 5.7880 |
| 2.6 | -0.2641 | 0.5462 | 1.1852 | 2.1048 | 2.8675 | 3.6967 | 5.8650 |
| 2.7 | -0.2674 | 0.5360 | 1.1755 | 2.1022 | 2.8753 | 3.7193 | 5.9406 |
| 2.8 | -0.2706 | 0.5259 | 1.1657 | 2.0991 | 2.8822 | 3.7408 | 6.0148 |
| 2.9 | -0.2734 | 0.5164 | 1.1562 | 2.0956 | 2.8880 | 3.7603 | 6.0841 |
| 3.0 | -0.2763 | 0.5060 | 1.1456 | 2.0913 | 2.8936 | 3.7806 | 6.1588 |
| 3.2 | -0.2809 | 0.4879 | 1.1266 | 2.0825 | 2.9014 | 3.8138 | 6.2872 |
| 3.4 | -0.2848 | 0.4706 | 1.1079 | 2.0724 | 2.9066 | 3.8427 | 6.4072 |
| 3.6 | -0.2879 | 0.4551 | 1.0905 | 2.0620 | 2.9094 | 3.8665 | 6.5133 |
| 3.8 | -0.2907 | 0.4395 | 1.0725 | 2.0503 | 2.9105 | 3.8882 | 6.6180 |
| 4.0 | -0.2929 | 0.4251 | 1.0554 | 2.0384 | 2.9098 | 3.9062 | 6.7126 |
| 4.5 | -0.2969 | 0.3924 | 1.0150 | 2.0070 | 2.9024 | 3.9401 | 6.9219 |
| 5.0 | -0.2991 | 0.3643 | 0.9784 | 1.9755 | 2.8893 | 3.9608 | 7.0937 |

Assuming the distribution of the logs is normal, the 10- and 100-year floods are computed using Equation (4-22) and Table 9 to be 13,945 CFS (395 CMS) and 35,965 CFS (1019 CMS), respectively. Using the computed skew of 0.236 and Table 11, the 10- and 100-year floods are 14,212 CFS (403 CMS) and 42,206 CFS (1196 CMS) respectively. Both the log-normal and skewed log-normal curves are plotted in Figure 28.

These actual flood data are also plotted on Log-Probability paper in Figure 28 together with the standard log-normal distributions. (Note: When plotting Q on the log scale, the actual values of Q are plotted rather than their logarithms since the log-scale effectively transforms the data to their respective logarithms.) Figure 28 shows the log-normal distributions fit the actual data better than the simple normal distribution shown in Figure 27.

Two useful relations are also available to approximate the mean and the standard deviation of the logarithms, $\bar{Q}_L$ and $S_L$, from $\bar{Q}$ and S of the original variables. These equations are

$$\bar{Q}_L = 1/2 \log \left(\frac{\bar{Q}^4}{\bar{Q}^2 + S^2}\right) \qquad (4\text{-}23)$$

and

$$S_L = \sqrt{\log \left(\frac{S^2 + \bar{Q}^2}{\bar{Q}^2}\right)} \qquad (4\text{-}24)$$

## 4.3.4 Gumbel Extreme Value Distribution

The Gumbel extreme value distribution, sometimes called the double exponential distribution of extreme values, can also be used to describe the distribution of hydrologic variables, especially peak discharges. It is based upon the assumption that the cumulative frequency distribution of the largest values of samples drawn from a large population can be described by the following equation

$$f(Q) = e^{-e^{-\alpha(Q-\beta)}} \qquad \text{Where} \quad \alpha = \frac{1.281}{S} \qquad (4\text{-}25)$$

$$\beta = \bar{Q} - 0.450\,S$$

Table 12. Example Computations for Log-Normal Frequency Distribution, Medina River, Texas

| Order | Flood, Q cfs | Log Q | $\frac{\text{Log } Q_i}{\bar{Q}_L}$ | $(\frac{\text{Log } Q_i}{\bar{Q}_L} - 1)$ | $(\frac{\text{Log } Q_i}{\bar{Q}_L} - 1)^2$ | $(\frac{\text{Log } Q_i}{\bar{Q}_L} - 1)^3$ |
|---|---|---|---|---|---|---|
| 1 | 31900 | 4.5038 | 1.2376 | .2376 | .05645 | .01341 |
| 2 | 31800 | 4.5024 | 1.2372 | .2372 | .05627 | .01335 |
| 3 | 17500 | 4.2430 | 1.1659 | .1659 | .02754 | .00457 |
| 4 | 17400 | 4.2406 | 1.1653 | .1653 | .02731 | .00451 |
| 5 | 14500 | 4.1614 | 1.1435 | .1435 | .02059 | .00295 |
| 6 | 13100 | 4.1173 | 1.1314 | .1314 | .01726 | .00227 |
| 7 | 12100 | 4.0828 | 1.1219 | .1219 | .01486 | .00181 |
| 8 | 9680 | 3.9859 | 1.0953 | .0953 | .00908 | .00086 |
| 9 | 9440 | 3.9750 | 1.0923 | .0923 | .00851 | .00079 |
| 10 | 9220 | 3.9647 | 1.0895 | .0895 | .00800 | .00072 |
| 11 | 8160 | 3.9117 | 1.0749 | .0749 | .00561 | .00042 |
| 12 | 7510 | 3.8756 | 1.0650 | .0650 | .00422 | .00027 |
| 13 | 6890 | 3.8382 | 1.0547 | .0547 | .00299 | .00016 |
| 14 | 6360 | 3.8035 | 1.0452 | .0451 | .00204 | .00009 |
| 15 | 5660 | 3.7528 | 1.0312 | .0312 | .00098 | .00003 |
| 16 | 5480 | 3.7388 | 1.0274 | .0274 | .00075 | .00002 |
| 17 | 5430 | 3.7348 | 1.0263 | .0263 | .00069 | .00002 |
| 18 | 5180 | 3.7143 | 1.0207 | .0207 | .00043 | .00001 |
| 19 | 4970 | 3.6955 | 1.0155 | .0155 | .00024 | ~ 0 |
| 20 | 4750 | 3.6768 | 1.0103 | .0103 | .00011 | ~ 0 |
| 21 | 4620 | 3.6646 | 1.007 | .0070 | .00005 | ~ 0 |
| 22 | 4130 | 3.6160 | .9936 | -.0064 | .00004 | ~ 0 |
| 23 | 3960 | 3.5977 | .9886 | -.0114 | .00013 | ~ 0 |
| 24 | 3540 | 3.549 | .9752 | -.0248 | .00061 | -.00002 |
| 25 | 3360 | 3.5263 | .9690 | -.0310 | .00096 | -.00003 |
| 26 | 3350 | 3.5250 | .9686 | -.0314 | .00098 | -.00003 |
| 27 | 3200 | 3.5052 | .9632 | -.0368 | .00136 | -.00005 |
| 28 | 3050 | 3.4843 | .9574 | -.0426 | .00181 | -.00008 |
| 29 | 2950 | 3.4698 | .9535 | -.0465 | .00217 | -.00010 |
| 30 | 2730 | 3.4362 | .9442 | -.0559 | .00311 | -.00017 |
| 31 | 2540 | 3.4048 | .9356 | -.0644 | .00415 | -.00027 |
| 32 | 2160 | 3.3345 | .9163 | -.0837 | .00701 | -.00059 |
| 33 | 2150 | 3.3324 | .9157 | -.0843 | .00710 | -.00060 |
| 34 | 2140 | 3.3304 | .9152 | -.0848 | .00720 | -.00061 |
| 35 | 2050 | 3.3118 | .9100 | -.0900 | .00809 | -.00073 |
| 36 | 2000 | 3.3010 | .9071 | -.0929 | .00863 | -.00080 |
| 37 | 1980 | 3.2967 | .9059 | -.0941 | .00857 | -.00083 |
| 38 | 1750 | 3.2430 | .8914 | -.1089 | .01185 | -.00129 |
| 39 | 1470 | 3.1673 | .8703 | -.1297 | .01681 | -.00218 |
| 40 | 1200 | 3.0792 | .8461 | -.1539 | .02368 | -.00364 |
| 41 | 890 | 2.9494 | .8105 | -.1895 | .03593 | -.00681 |
| 42 | 865 | 2.9370 | .9071 | -.1929 | .03723 | -.00718 |
| 43 | 801 | 2.9036 | .7979 | -.2021 | .04085 | -.00826 |
| TOTALS | | 156.4840 | | | .4925 | .01259 |

Table 12. Example Computations for Log-Normal Frequency Distribution, Medina River, Texas (Continued)

$$\bar{Q}_L = \frac{\sum_{i=1}^{n} \text{Log } Q_i}{n} = \frac{156.4840}{43} = 3.6392$$

$$S_L = \bar{Q}_L \sqrt{\frac{\sum_{i=1}^{n}\left(\frac{\text{Log } Q_i}{\bar{Q}_L} - 1\right)^2}{n-1}} = 3.6392\sqrt{\frac{0.4925}{42}} = 0.3941$$

$$V_L = \frac{S_L}{\bar{Q}_L} = \frac{0.3941}{3.6392} = 0.108$$

$$G_L = \frac{\sum_{i=1}^{n}\left(\frac{\text{Log } Q_i}{\bar{Q}_L} - 1\right)^3}{(n-1)\ V^3} = \frac{.01259}{42\,(0.1083)^3} = 0.236$$

In a manner analogous to that of the normal distribution, values of the distribution function can be computed from Equation (4-25). These values are tabulated for convenience in Table 13.

Characteristics of the Gumbel Extreme Value Distribution are that the mean flow, Q, occurs at the return period of Tr = 2.33 years and that it has a positive skew, i.e. it is skewed towards the high flows or extreme values.

As was the case with the two previous distributions, special probability paper (called Gumbel Paper) has been developed so that sample data, if it is distributed according to Equation (4-25), will plot as a straight line. This paper is not available commercially, but most USGS offices have prepared forms of this paper on which the horizontal scale has been transformed by the double logarithmic transform of Equation (4-25).

Peak flow data for the Medina River, Table 10, can be fit with a Gumbel distribution using Equation (4-21) and values of K from Table 13. The 10- and 100-year floods computed from the Gumbel distribution are 17,115 CFS (484.7 CMS) and 31,604 CFS (895.0 CMS), respectively, as shown in Figure 29. Also plotted on the Gumbel graph paper in Figure 29 are the actual flood data.

Although the Gumbel Distribution is skewed positively, it does not account directly for the computed skew of the data but does predict the high flows reasonably well. However, the entire curve fit is not much better than that obtained with the normal distribution indicating this peak flow series is not distributed according to the double exponential distribution of Equation (4-25).

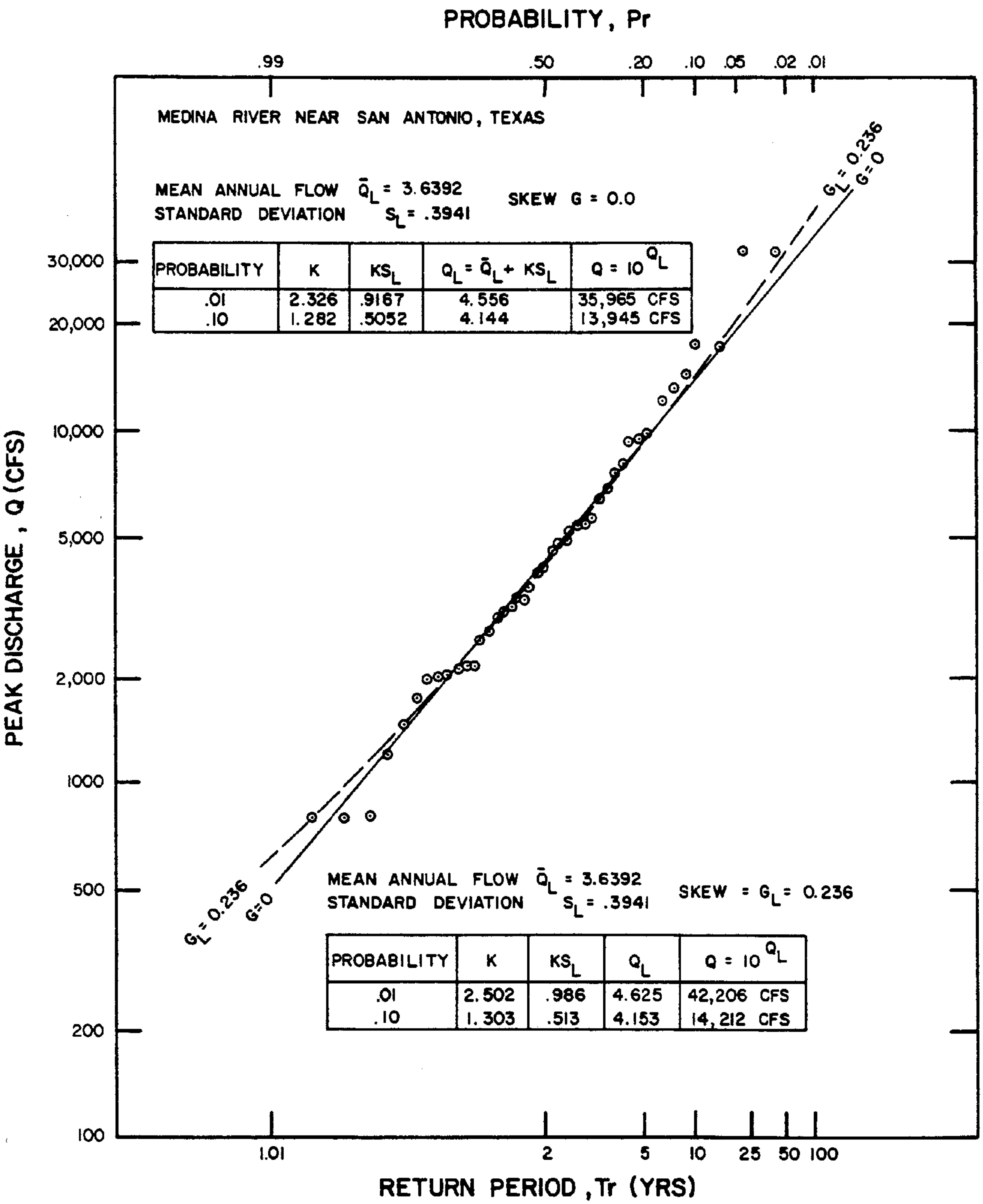



Figure 28. Log-Normal Frequency Distribution Analysis, Medina River, Texas

Table 13. Cumulative Distribution Function for Gumbel Extreme Value Distribution

| Sample Size n | Exceedance Probability in % | | | | | | |
|---|---|---|---|---|---|---|---|
| | 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
| | Corresponding Return Period in Years | | | | | | |
| | 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 10 | -0.1355 | 1.0581 | 1.8483 | 2.8468 | 3.5876 | 4.3228 | 6.0219 |
| 15 | -0.1433 | 0.9672 | 1.7025 | 2.6315 | 3.3207 | 4.0048 | 5.5857 |
| 20 | -0.1478 | 0.9186 | 1.6247 | 2.5169 | 3.1787 | 3.8357 | 5.3538 |
| 25 | -0.1506 | 0.8879 | 1.5755 | 2.4442 | 3.0887 | 3.7285 | 5.2068 |
| 30 | -0.1525 | 0.8664 | 1.5410 | 2.3933 | 3.0257 | 3.6533 | 5.1038 |
| 35 | -0.1540 | 0.8504 | 1.5153 | 2.3555 | 2.9789 | 3.5976 | 5.0273 |
| 40 | -0.1552 | 0.8379 | 1.4955 | 2.3262 | 2.9426 | 3.5543 | 4.9680 |
| 45 | -0.1561 | 0.8280 | 1.4795 | 2.3027 | 2.9134 | 3.5196 | 4.9204 |
| 50 | -0.1568 | 0.8197 | 1.4662 | 2.2831 | 2.8892 | 3.4907 | 4.8808 |
| 55 | -0.1574 | 0.8128 | 1.4552 | 2.2668 | 2.8690 | 3.4667 | 4.8478 |
| 60 | -0.1580 | 0.8069 | 1.4457 | 2.2529 | 2.8517 | 3.4460 | 4.8195 |
| 65 | -0.1584 | 0.8019 | 1.4377 | 2.2410 | 2.8369 | 3.4285 | 4.7955 |
| 70 | -0.1588 | 0.7973 | 1.4304 | 2.2302 | 2.8236 | 3.4126 | 4.7738 |
| 75 | -0.1592 | 0.7934 | 1.4242 | 2.2211 | 2.8123 | 3.3991 | 4.7552 |
| 80 | -0.1595 | 0.7899 | 1.4186 | 2.2128 | 2.8020 | 3.3869 | 4.7384 |
| 85 | -0.1598 | 0.7868 | 1.4135 | 2.2054 | 2.7928 | 3.3759 | 4.7234 |
| 90 | -0.1600 | 0.7840 | 1.4090 | 2.1987 | 2.7845 | 3.3660 | 4.7098 |
| 95 | -0.1602 | 0.7815 | 1.4049 | 2.1926 | 2.7770 | 3.3570 | 4.6974 |
| 100 | -0.1604 | 0.7791 | 1.4011 | 2.1869 | 2.7699 | 3.3487 | 4.6860 |

### 4.3.5 Log-Pearson Type III Distribution

Another distribution which has found wide application in hydrologic analysis is the log-Pearson Type III distribution. The log-Pearson Type III distribution is a three parameter gamma distribution with a logarithmic transform of the independent variable. It is one of a number of standard distributions which have been developed, more or less empirically, which can be applied to statistical problems. Its use is based simply on the fact that it very often fits the available data quite well, and it is flexible enough to be used with a wide variety of distributions. It is this flexibility which has lead the U.S. Water Resources Council to recommend its use as the standard distribution for flood frequency studies by all U.S. Government agencies.

The log-Pearson III distribution differs from most of the distributions discussed above in that the three parameters, mean flow, standard deviation and coefficient of skew are necessary to describe the distribution. By judicious selection of these three parameters, it is possible to fit just about any

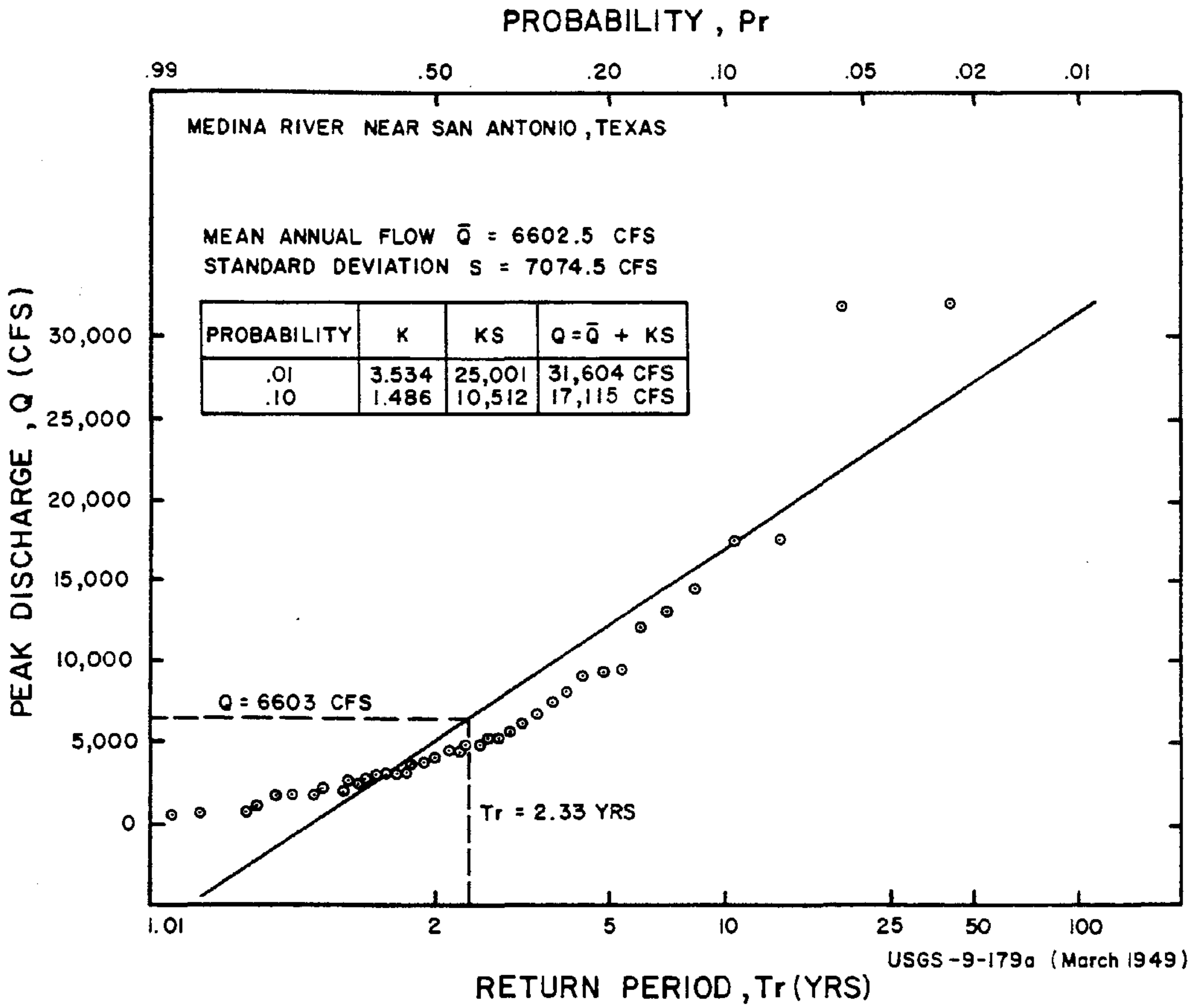



Figure 29. Gumbel Extreme Value Frequency Distribution Analysis, Medina River, Texas

shape of distribution. An extensive treatment on the use of this distribution in the determination of flood frequency distributions is presented in Bulletin 17B, "Guidelines for Determining Flood Frequency" by the U.S. Water Resources Council, revised September, 1981.

An abbreviated Table of the log-Pearson III Distribution Functions is given in Table 14. (Extensive tables which reduce the amount of interpolation can be found in Bulletin 17B). Using the mean, standard deviation and skew coefficient for any set of log-transformed annual peak flow data, in conjunction with Table 14, the flood with any exceedance frequency can be computed from the equation

$$\log Q = \bar{Q}_L + KS_L \tag{4-26}$$

where $\bar{Q}_L$ and $S_L$ are as previously defined and K is a function of both the standard deviation and the coefficient of skew.

Table 14. Cumulative Distribution Function for Log-Pearson Type III Distribution

| Coef. of Skew | Exceedance Probability in % | | | | | | |
|---|---|---|---|---|---|---|---|
| | 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
| | Corresponding Return Period in Years | | | | | | |
| | 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 3.0 | -0.3955 | 0.4204 | 1.1801 | 2.2778 | 3.1519 | 4.0514 | 6.2051 |
| 2.8 | -0.3835 | 0.4598 | 1.2101 | 2.2747 | 3.1140 | 3.9730 | 6.0186 |
| 2.6 | -0.3685 | 0.4987 | 1.2377 | 2.2674 | 3.0712 | 3.8893 | 5.6282 |
| 2.4 | -0.3506 | 0.5368 | 1.2624 | 2.2558 | 3.0233 | 3.8001 | 5.6282 |
| 2.2 | -0.3300 | 0.5738 | 1.2841 | 2.2397 | 2.9703 | 3.7054 | 5.4243 |
| 2.0 | -0.3069 | 0.6094 | 1.3026 | 2.2189 | 2.9120 | 3.6052 | 5.2146 |
| 1.8 | -0.2815 | 0.6434 | 1.3176 | 2.1933 | 2.8485 | 3.4994 | 4.9994 |
| 1.6 | -0.2542 | 0.6753 | 1.3290 | 2.1629 | 2.7796 | 3.3880 | 4.7788 |
| 1.4 | -0.2254 | 0.7051 | 1.3367 | 2.1277 | 2.7056 | 3.2713 | 4.5530 |
| 1.2 | -0.1952 | 0.7326 | 1.3405 | 2.0876 | 2.6263 | 3.1494 | 4.3226 |
| 1.0 | -0.1640 | 0.7575 | 1.3404 | 2.0427 | 2.5421 | 3.0226 | 4.0880 |
| 0.8 | -0.1320 | 0.7799 | 1.3364 | 1.9931 | 2.4530 | 2.8910 | 3.8498 |
| 0.6 | -0.0995 | 0.7995 | 1.3285 | 1.9390 | 2.3593 | 2.7551 | 3.6087 |
| 0.4 | -0.0665 | 0.8164 | 1.3167 | 1.8804 | 2.2613 | 2.6154 | 3.3657 |
| 0.2 | -0.0333 | 0.8304 | 1.3011 | 1.8176 | 2.1594 | 2.4723 | 3.1217 |
| 0.0 | 0.0000 | 0.8416 | 1.2816 | 1.7507 | 2.0538 | 2.3264 | 2.8782 |
| -0.2 | 0.0333 | 0.8499 | 1.2582 | 1.6800 | 1.9450 | 2.1784 | 2.6367 |
| -0.4 | 0.0665 | 0.8551 | 1.2311 | 1.6057 | 1.8336 | 2.0293 | 2.3994 |
| -0.6 | 0.0995 | 0.8572 | 1.2003 | 1.5283 | 1.7203 | 1.8803 | 2.1688 |
| -0.8 | 0.1320 | 0.8561 | 1.1657 | 1.4481 | 1.6060 | 1.7327 | 1.9481 |
| -1.0 | 0.1640 | 0.8516 | 1.1276 | 1.3658 | 1.4919 | 1.5884 | 1.7406 |
| -1.2 | 0.1952 | 0.8437 | 1.0861 | 1.2823 | 1.3793 | 1.4494 | 1.5502 |
| -1.4 | 0.2254 | 0.8322 | 1.0414 | 1.1984 | 1.2700 | 1.3182 | 1.3798 |
| -1.6 | 0.2542 | 0.8172 | 0.9942 | 1.1157 | 1.1658 | 1.1968 | 1.2313 |
| -1.8 | 0.2815 | 0.7986 | 0.9450 | 1.0354 | 1.0686 | 1.0871 | 1.1047 |
| -2.0 | 0.3069 | 0.7769 | 0.8946 | 0.9592 | 0.9798 | 0.9900 | 0.9980 |
| -2.2 | 0.3300 | 0.7521 | 0.8442 | 0.8881 | 0.9001 | 0.9052 | 0.9085 |
| -2.4 | 0.3506 | 0.7250 | 0.7947 | 0.8232 | 0.8296 | 0.8320 | 0.8332 |
| -2.6 | 0.3685 | 0.6960 | 0.7471 | 0.7646 | 0.7678 | 0.7688 | 0.7692 |
| -2.8 | 0.3835 | 0.6660 | 0.7021 | 0.7123 | 0.7138 | 0.7142 | 0.7143 |
| -3.0 | 0.3955 | 0.6357 | 0.6602 | 0.6659 | 0.6665 | 0.6667 | 0.6667 |

from WRC, 1981

Again, it would be possible to develop special probability paper, so that the log-Pearson III distribution would plot as a straight line. However, the log-Pearson III distribution has varying shape statistics, i.e. $K = f(S_L, G_L)$ so that a separate probability paper would be required for each different distribution. Since this is impractical, log-Pearson III distributions are usually plotted on log-normal probability graph paper even though the plotted frequency distribution may not be a straight line.

Table 14 and Equation (4-26) are used to compute the log-Pearson III distribution for the 10- and 100-year flood using the parameters, $Q_L$, $S_L$, and $G_L$ for the Medina River flood data of Table 12. (To help define the distribution, the 25- and 50-year floods have also been computed). Using the station skew of 0.236, the log-Pearson III distribution estimates the 10- and 100- year floods at 14,226 CFS (403 CMS) and 42,042 CFS (1191 CMS), respectively. The log-Pearson III distribution ($G_L = 0.236$) together with the actual data from Table 10 are plotted in Figure 30 on log-normal probability paper.

Bulletin 17B outlines three methods for selection of the skew coefficient. These include the station skew, a generalized skew and a weighted skew. Since the skew coefficient is very sensitive to extreme values, the station skew, or the skew coefficient computed from the actual data may not be accurate if the sample is a short record. In this case, Bulletin 17B recommends use of a generalized skew coefficient determined from a map giving generalized skew coefficients of the logarithms of annual maximum streamflows throughout the United States. This map also gives average skew coefficients by one degree quadrangles over most of the country.

The generalized skew coefficient for the Medina River is -0.252. Using this option, the 10- and 100-year floods for the Medina River are estimated from Equation (4-26) to be 13,564 CFS (384.1 CMS) and 30,411 4FS (861.2 CMS), respectively. This log-Pearson III distribution (generalized skew coefficient, $G_L = -0.252$) is also plotted on Figure 30.

Often the station skew and generalized skew can be combined to provide a better estimate for a given sample of flood data. Bulletin 17B outlines a procedure based on the concept that the "mean-square error (MSE) of the weighted estimate is minimized by weighting the station and generalized skews in inverse proportion to their individual mean-square errors". The mean-square error is defined as the sum of the squared differences between the true and estimated values of a quantity divided by the number of observations. In analytical form, this concept is given by the equation

$$G_W = \frac{MSE_{\bar{G}_L}(G_L) + MSE_{G_L}(\bar{G}_L)}{MSE_{\bar{G}_L} + MSE_{G_L}} \qquad (4\text{-}27)$$

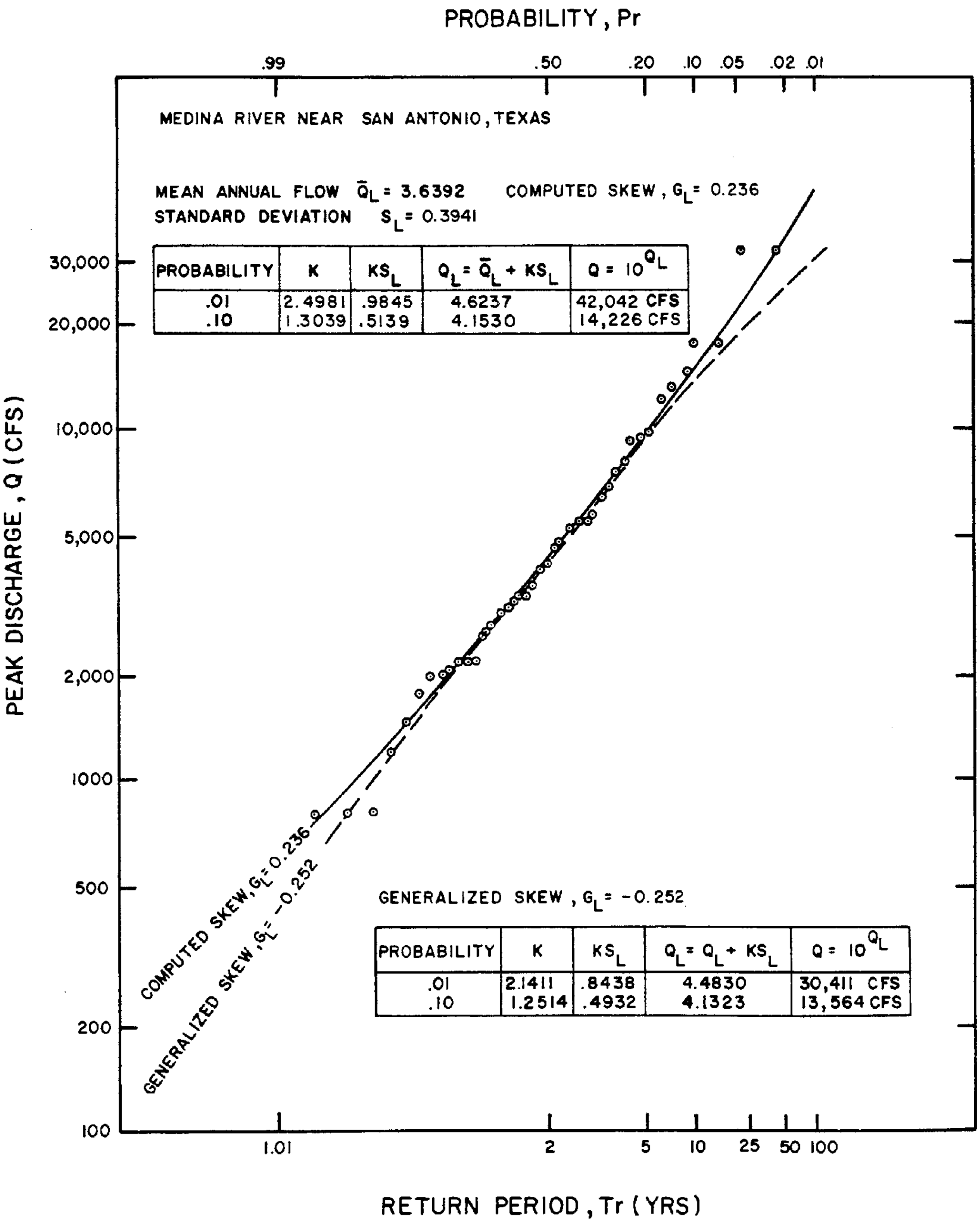



Figure 30. Log Pearson Type III Distribution Analysis, Medina River, Texas

where $G_w$ is the weighted skew, $G_L$ is the station skew, $\overline{G}_L$ is the generalized skew, and $MSE_{G_L}$ and $MSE_{\overline{G}_L}$ are the mean square errors for the station and generalized skews, respectively.

When $\overline{G}_L$ is taken from the map of generalized skews in Bulletin 17B, $MSE_{\overline{G}_L} = 0.302$. The value of $MSE_{G_L}$ can be obtained from Table 15 taken directly from Bulletin 17B or approximated by the equation

$$MSE_{G_L} = 10^{\left\{A - B\left[\log_{10}\left(\frac{n}{10}\right)\right]\right\}} \qquad (4\text{-}28)$$

where n is the period of record,

$A = -0.33 + 0.08\,|G_L|$ for $|G_L| \leq 0.90$
$A = -0.52 + 0.30\,|G_L|$ for $|G_L| \leq 0.90$

and

$B = 0.94 - 0.26\,|G_L|$ for $|G_L| \leq 1.50$
$B = 0.55$ for $|G_L| \leq 1.50$

To illustrate the determination of a weighted skew, consider the Medina River data used in the above illustrations. For these data, the station and generalized skews have already been determined to be $G_L = 0.236$ and $\overline{G}_L = -0.252$, respectively. The mean-square error of $\overline{G}_L$, $MSE_{\overline{G}_L}$, is 0.302 and from Equation (4-28) $MSE_{G_L} = 0.136$. From Equation (4-27), the weighted skew is computed as

$$G_w = \frac{0.302\,(0.236) + 0.136\,(-0.252)}{0.302 + 0.136} = 0.085$$

If the difference between the generalized and station skews is greater than 0.5, the data and basin characteristics should be reviewed, possibly giving more weight to the station skew.

The USGS has developed Program J407, an example output from which is shown in Table 16, for statistical flood frequency analysis of annual peak flow records. The analysis follows WRC Bulletin 17B guidelines including the calculation of a log-Pearson III frequency curve based on the mean, standard deviation and skewness of the logarithms of the recorded annual peak flows.

Table 15. Summary of Mean Square Error of Station Skew as a Function of Record Length and Station Skew

| STATION SKEW ($G_L$ OR $\bar{G}_L$) | RECORD LENGTH, IN YEARS | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 | 90 | 100 |
| 0.0 | 0.468 | 0.244 | 0.167 | 0.127 | 0.103 | 0.087 | 0.075 | 0.066 | 0.059 | 0.054 |
| 0.1 | 0.476 | 0.253 | 0.175 | 0.134 | 0.109 | 0.093 | 0.080 | 0.071 | 0.064 | 0.058 |
| 0.2 | 0.485 | 0.262 | 0.183 | 0.142 | 0.116 | 0.099 | 0.086 | 0.077 | 0.069 | 0.063 |
| 0.3 | 0.494 | 0.272 | 0.192 | 0.150 | 0.123 | 0.105 | 0.092 | 0.082 | 0.074 | 0.068 |
| 0.4 | 0.504 | 0.282 | 0.201 | 0.158 | 0.131 | 0.113 | 0.099 | 0.089 | 0.080 | 0.073 |
| 0.5 | 0.513 | 0.293 | 0.211 | 0.167 | 0.139 | 0.120 | 0.106 | 0.095 | 0.087 | 0.079 |
| 0.6 | 0.522 | 0.303 | 0.221 | 0.176 | 0.148 | 0.128 | 0.114 | 0.102 | 0.093 | 0.086 |
| 0.7 | 0.532 | 0.315 | 0.231 | 0.186 | 0.157 | 0.137 | 0.122 | 0.110 | 0.101 | 0.093 |
| 0.8 | 0.542 | 0.326 | 0.243 | 0.196 | 0.167 | 0.146 | 0.130 | 0.118 | 0.109 | 0.100 |
| 0.9 | 0.562 | 0.345 | 0.259 | 0.211 | 0.181 | 0.159 | 0.142 | 0.130 | 0.119 | 0.111 |
| 1.0 | 0.603 | 0.376 | 0.285 | 0.235 | 0.202 | 0.178 | 0.160 | 0.147 | 0.135 | 0.126 |
| 1.1 | 0.646 | 0.410 | 0.315 | 0.261 | 0.225 | 0.200 | 0.181 | 0.166 | 0.153 | 0.143 |
| 1.2 | 0.692 | 0.448 | 0.347 | 0.290 | 0.252 | 0.225 | 0.204 | 0.187 | 0.174 | 0.163 |
| 1.3 | 0.741 | 0.488 | 0.383 | 0.322 | 0.281 | 0.252 | 0.230 | 0.212 | 0.197 | 0.185 |
| 1.4 | 0.794 | 0.533 | 0.422 | 0.357 | 0.314 | 0.283 | 0.259 | 0.240 | 0.224 | 0.211 |
| 1.5 | 0.851 | 0.581 | 0.465 | 0.397 | 0.351 | 0.318 | 0.292 | 0.271 | 0.254 | 0.240 |
| 1.6 | 0.912 | 0.623 | 0.498 | 0.425 | 0.376 | 0.340 | 0.313 | 0.291 | 0.272 | 0.257 |
| 1.7 | 0.977 | 0.667 | 0.534 | 0.456 | 0.403 | 0.365 | 0.335 | 0.311 | 0.292 | 0.275 |
| 1.8 | 1.047 | 0.715 | 0.572 | 0.489 | 0.432 | 0.391 | 0.359 | 0.334 | 0.313 | 0.295 |
| 1.9 | 1.122 | 0.766 | 0.613 | 0.523 | 0.463 | 0.419 | 0.385 | 0.358 | 0.335 | 0.316 |
| 2.0 | 1.202 | 0.821 | 0.657 | 0.561 | 0.496 | 0.449 | 0.412 | 0.383 | 0.359 | 0.339 |
| 2.1 | 1.288 | 0.880 | 0.704 | 0.601 | 0.532 | 0.481 | 0.442 | 0.410 | 0.385 | 0.363 |
| 2.2 | 1.380 | 0.943 | 0.754 | 0.644 | 0.570 | 0.515 | 0.473 | 0.440 | 0.412 | 0.389 |
| 2.3 | 1.479 | 1.010 | 0.808 | 0.690 | 0.610 | 0.552 | 0.507 | 0.471 | 0.442 | 0.417 |
| 2.4 | 1.585 | 1.083 | 0.866 | 0.739 | 0.654 | 0.592 | 0.543 | 0.505 | 0.473 | 0.447 |
| 2.5 | 1.698 | 0.160 | 0.928 | 0.792 | 0.701 | 0.634 | 0.582 | 0.541 | 0.507 | 0.479 |
| 2.6 | 1.820 | 1.243 | 0.994 | 0.849 | 0.751 | 0.679 | 0.624 | 0.580 | 0.543 | 0.513 |
| 2.7 | 1.950 | 1.332 | 1.066 | 0.910 | 0.805 | 0.728 | 0.669 | 0.621 | 0.582 | 0.550 |
| 2.8 | 2.089 | 1.427 | 1.142 | 0.975 | 0.862 | 0.780 | 0.716 | 0.666 | 0.624 | 0.589 |
| 2.9 | 2.239 | 1.529 | 1.223 | 1.044 | 0.924 | 0.836 | 0.768 | 0.713 | 0.669 | 0.631 |
| 3.0 | 2.399 | 1.638 | 1.311 | 1.119 | 0.990 | 0.895 | 0.823 | 0.764 | 0.716 | 0.676 |

from WRC, 1981

Table 16. Sample Output, USGS Program J407 for Log-Pearson Type III Frequency Distribution

PGM J407 VER 3.7
(REV 11/5/81)

U. S. GEOLOGICAL SURVEY
ANNUAL PEAK FLOW FREQUENCY ANALYSIS
FOLLOWING WRC GUIDELINES BULL. 17-B.

PK FLOW FREQUENCY ANALYSIS
RUN-DATE 5/21/84 AT 2015 SEQ 1.0001

OPTIONS IN EFFECT -- PLOT NOWC LGPT NOUH PPOS NORS NOEX CLIM

STATION - 08181500 /USGS MEDINA RIVER AT SAN ANTONIO, TEX. 1940-1982 08181500 /USGS

I N P U T D A T A S U M M A R Y

| -- YEARS OF RECORD -- SYSTEMATIC | HISTORIC | HISTORIC PEAKS | GENERALIZED SKEW | STD. ERROR OF GENERAL. SKEW | SKEW OPTION | GAGE BASE DISCHARGE | USER-SET OUTLIER CRITERIA HIGH OUTLIER | LOW OUTLIER |
|---|---|---|---|---|---|---|---|---|
| 43 | 0 | 0 | -0.252 | -- | WRC WEIGHTED | 0.0 | -- | -- |

********* NOTICE -- PRELIMINARY MACHINE COMPUTATIONS. *********
********* USER RESPONSIBLE FOR ASSESSMENT AND INTERPRETATION. *********

*WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE. 0.0
*WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION. 372.6
*WCF163I-NO HIGH OUTLIERS OR HISTORIC PEAKS EXCEEDED HHBASE. 509[illegible].9

ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

| | FLOOD BASE DISCHARGE | FLOOD BASE EXCEEDANCE PROBABILITY | LOGARITHMIC MEAN | LOGARITHMIC STANDARD DEVIATION | LOGARITHMIC SKEW |
|---|---|---|---|---|---|
| SYSTEMATIC RECORD | 0.0 | 1.0000 | 3.6392 | 0.3941 | 0.236 |
| W R C ESTIMATE | 0.0 | 1.0000 | 3.6392 | 0.3941 | 0.085 |

ANNUAL FREQUENCY CURVE ORDINATES -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

| ANNUAL EXCEEDANCE PROBABILITY | W R C ESTIMATE | SYSTEMATIC RECORD | 'EXPECTED PROBABILITY' ESTIMATE | 95-PCT CONFIDENCE LIMITS FOR W R C ESTIMATES LOWER | UPPER |
|---|---|---|---|---|---|
| 0.9950 | 452.4 | 514.6 | -- | 259.7 | 677.8 |
| 0.9900 | 558.6 | 618.4 | -- | 334.0 | 814.7 |
| 0.9500 | 1001.4 | 1043.4 | -- | 606.8 | 1363.6 |
| 0.9000 | 1373.5 | 1396.8 | -- | 964.3 | 1811.6 |
| 0.8000 | 2022.9 | 2012.7 | -- | 1502.7 | 2587.4 |
| 0.5000 | 4301.2 | 4204.2 | -- | 3411.1 | 5419.0 |
| 0.2000 | 9313.2 | 9236.5 | -- | 7284.7 | 12524.3 |
| 0.1000 | 14049.3 | 14227.0 | -- | 10635.4 | 20063.8 |
| 0.0400 | 21901.7 | 22910.1 | -- | 15828.6 | 33745.5 |
| 0.0200 | 29266.6 | 31441.1 | -- | 20440.5 | 47570.2 |
| 0.0100 | 38063.5 | 42045.3 | -- | 25720.8 | 65060.3 |
| 0.0050 | 48496.1 | 55129.2 | -- | 31765.7 | 86924.8 |
| 0.0020 | 65186.0 | 77044.1 | -- | 41046.4 | 123958.4 |

Table 16. Sample Output, USGS Program J407 for Log-Pearson Type III Frequency Distribution (Continued)

PGM J407 VER 3.7 U. S. GEOLOGICAL SURVEY
(REV 11/5/81) ANNUAL PEAK FLOW FREQUENCY ANALYSIS PK FLOW FREQUENCY ANALYSIS
FOLLOWING WRC GUIDELINES BULL. 17-B. RUN-DATE 5/21/84 AT 2015 SEQ 1.0001

STATION - 08181500 /USGS MEDINA RIVER AT SAN ANTONIO, TEX. 1940-1982 08181500 /USGS

********* NOTICE -- PRELIMINARY MACHINE COMPUTATIONS. *********
********* USER RESPONSIBLE FOR ASSESSMENT AND INTERPRETATION. *********

INPUT DATA LISTING — EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

| WATER YEAR | DISCHARGE | CODES | WATER YEAR | RANKED DISCHARGE | SYSTEMATIC RECORD | W R C ESTIMATE |
|---|---|---|---|---|---|---|
| 1940 | 2540.0 | K | 1973 | 31900.0 | 0.0227 | 0.0227 |
| 1941 | 6890.0 | K | 1946 | 31800.0 | 0.0455 | 0.0455 |
| 1942 | 17500.0 | K | 1942 | 17500.0 | 0.0682 | 0.0682 |
| 1943 | 12100.0 | K | 1949 | 17400.0 | 0.0909 | 0.0909 |
| 1944 | 2000.0 | K | 1981 | 14500.0 | 0.1136 | 0.1136 |
| 1945 | 3540.0 | K | 1968 | 13100.0 | 0.1364 | 0.1364 |
| 1946 | 31800.0 | K | 1943 | 12100.0 | 0.1591 | 0.1591 |
| 1947 | 1470.0 | K | 1974 | 9680.0 | 0.1818 | 0.1818 |
| 1948 | 2050.0 | K | 1978 | 9440.0 | 0.2045 | 0.2045 |
| 1949 | 17400.0 | K | 1958 | 9220.0 | 0.2273 | 0.2273 |
| 1950 | 5660.0 | K | 1982 | 8160.0 | 0.2500 | 0.2500 |
| 1951 | 2150.0 | K | 1976 | 7510.0 | 0.2727 | 0.2727 |
| 1952 | 801.0 | K | 1941 | 6890.0 | 0.2955 | 0.2955 |
| 1953 | 4960.0 | K | 1972 | 6360.0 | 0.3182 | 0.3182 |
| 1954 | 865.0 | K | 1950 | 5660.0 | 0.3409 | 0.3409 |
| 1955 | 1200.0 | K | 1967 | 5480.0 | 0.3636 | 0.3636 |
| 1956 | 1750.0 | K | 1965 | 5430.0 | 0.3864 | 0.3864 |
| 1957 | 5180.0 | K | 1957 | 5180.0 | 0.4091 | 0.4091 |
| 1958 | 9220.0 | K | 1953 | 4960.0 | 0.4318 | 0.4318 |
| 1959 | 3350.0 | K | 1979 | 4750.0 | 0.4545 | 0.4545 |
| 1960 | 3200.0 | K | 1977 | 4620.0 | 0.4773 | 0.4773 |
| 1961 | 3050.0 | K | 1975 | 4130.0 | 0.5000 | 0.5000 |
| 1962 | 3960.0 | K | 1962 | 3960.0 | 0.5227 | 0.5227 |
| 1963 | 890.0 | K | 1945 | 3540.0 | 0.5455 | 0.5455 |
| 1964 | 2140.0 | K | 1970 | 3360.0 | 0.5682 | 0.5682 |
| 1965 | 5430.0 | K | 1959 | 3350.0 | 0.5909 | 0.5909 |
| 1966 | 2160.0 | K | 1960 | 3200.0 | 0.6136 | 0.6136 |
| 1967 | 5480.0 | K | 1961 | 3050.0 | 0.6364 | 0.6364 |
| 1968 | 13100.0 | K | 1971 | 2950.0 | 0.6591 | 0.6591 |
| 1969 | 2730.0 | K | 1969 | 2730.0 | 0.6818 | 0.6818 |
| 1970 | 3360.0 | K | 1940 | 2540.0 | 0.7045 | 0.7045 |
| 1971 | 2950.0 | K | 1966 | 2160.0 | 0.7273 | 0.7273 |
| 1972 | 6360.0 | K | 1951 | 2150.0 | 0.7500 | 0.7500 |
| 1973 | 31900.0 | K | 1964 | 2140.0 | 0.7727 | 0.7727 |
| 1974 | 9680.0 | K | 1948 | 2050.0 | 0.7955 | 0.7955 |

-- CONTINUED --

## Table 16. Sample Output, USGS Program J407 for Log-Pearson Type III Frequency Distribution (Continued)

```
PGM J407 VER 3.7                      U. S. GEOLOGICAL SURVEY
(REV 11/5/81)               ANNUAL  PEAK  FLOW  FREQUENCY  ANALYSIS           PK FLOW FREQUENCY ANALYSIS
                             FOLLOWING WRC GUIDELINES BULL. 17-B.              RUN-DATE  5/21/84 AT 2015   SEQ  1.0001

  STATION -  08181500      /USGS    MEDINA RIVER AT SAN ANTONIO, TEX.          1940-1982            08181500     /USGS

                 *********    NOTICE  --  PRELIMINARY MACHINE COMPUTATIONS.       *********
                 *********  USER RESPONSIBLE FOR ASSESSMENT AND INTERPRETATION.  *********
```

INPUT DATA LISTING / EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

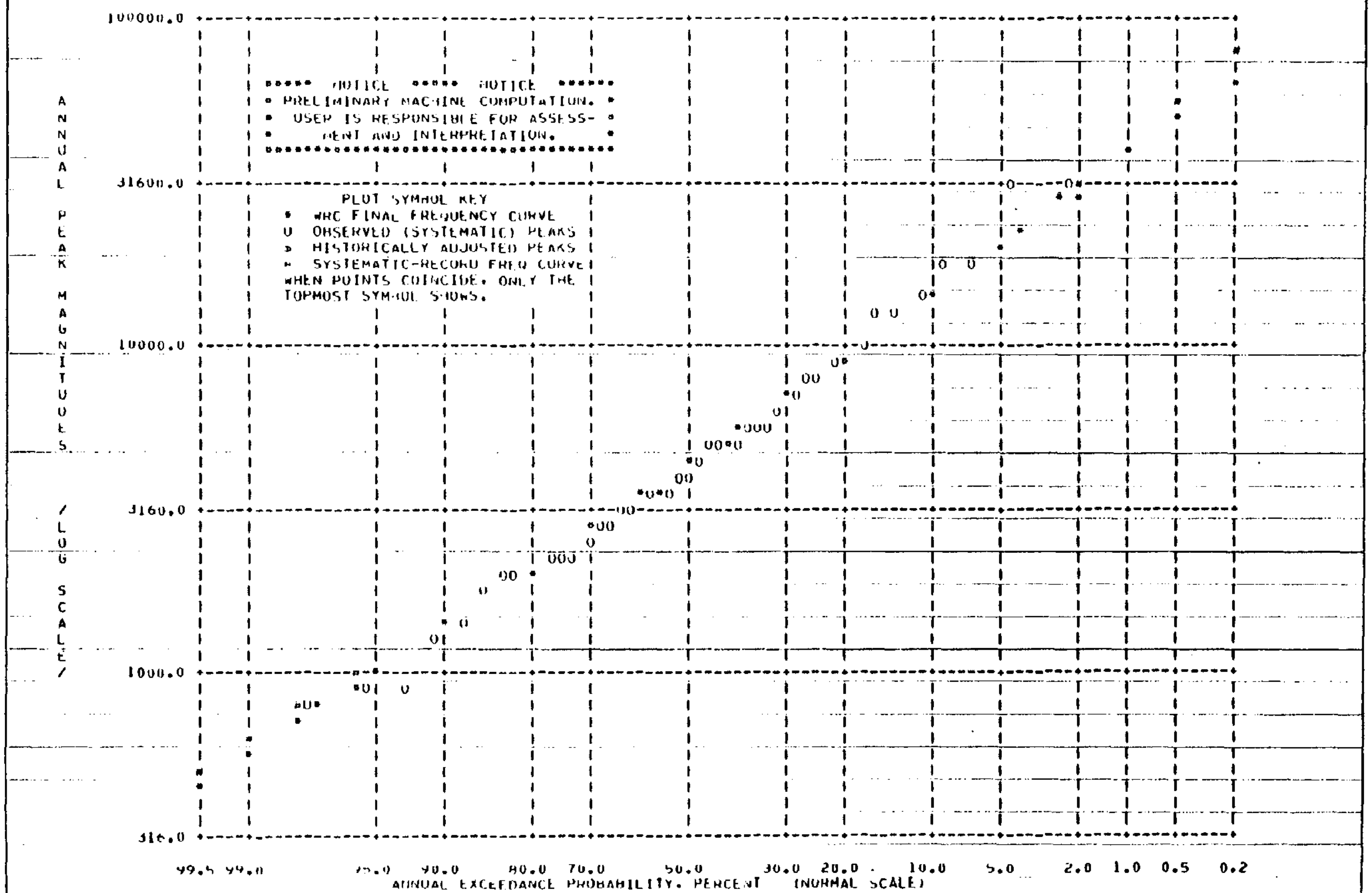
-- CONTINUED --

| WATER YEAR | DISCHARGE | CODES | WATER YEAR | RANKED DISCHARGE | SYSTEMATIC RECORD | W R C ESTIMATE |
|---|---|---|---|---|---|---|
| 1975 | 4130.0 | K | 1944 | 2000.0 | 0.8182 | 0.8182 |
| 1976 | 7510.0 | K | 1980 | 1980.0 | 0.8409 | 0.8409 |
| 1977 | 4620.0 | K | 1956 | 1750.0 | 0.8636 | 0.8636 |
| 1978 | 9440.0 | K | 1947 | 1470.0 | 0.8864 | 0.8864 |
| 1979 | 4750.0 | K | 1955 | 1200.0 | 0.9091 | 0.9091 |
| 1980 | 1980.0 | | 1963 | 890.0 | 0.9318 | 0.9318 |
| 1981 | 14500.0 | K | 1954 | 865.0 | 0.9545 | 0.9545 |
| 1982 | 8160.0 | | 1952 | 801.0 | 0.9773 | 0.9773 |

Table 16. Sample Output, USGS Program J407 for Log-Pearson Type III Frequency Distribution (Continued)

```
PGM J407 VER 3.7                     U. S. GEOLOGICAL SURVEY
(REV 11/5/81)               ANNUAL PEAK FLOW FREQUENCY ANALYSIS          PK FLOW FREQUENCY ANALYSIS
                            FOLLOWING WRC GUIDELINES BULL. 17-B.          RUN-DATE 5/21/84 AT 2015   SEQ 1.0001

 STATION - 08181500    /USGS    MEDINA RIVER AT SAN ANTONIO, TEX.          1940-1982          08181500      /USGS
```



Peak flow data are taken from WATSTORE with the peak flow file retrieval program discussed in Section 3.0. In addition to the basic frequency analysis, Program J407 allows for adjustments for zero flows, peaks below gage base, low and high outliers, historic information and regional skew information. The program also contains an option to include a printer plot of the expected frequency curve.

To illustrate the output from Program J407, a log-Pearson III frequency analysis was performed on the Medina River data using the Bulletin 17B option for weighted skew. The output shown on Table 16 includes Input Data Summary, Annual Frequency Curve Parameters, Discharge at Selected Exceedance Probabilities, Input Data Listing, Data for Plotting Positions, and a printer plot of the frequency distribution curve including the observed flow peaks. Using the weighted skew of 0.085, the 10- and 100- year floods are estimated as 14,049 CFS (397.9 CMS) and 38,064 CFS (1078.0 CMS) respectively.

Presently, this information can be obtained from any USGS District office for peak flow data in WATSTORE. It is expected that this output will also be obtainable from USGS sub-District offices in the near future and also can be obtained by anyone with access to WATSTORE.

### 4.3.6 Evaluation of Flood Frequency Predictions

The peak flow data for the Medina River gage have now been analyzed by four different standard frequency distributions, and in the case of log-Pearson III distribution by three different options for the inclusion of skewness. The predicted 10-year and 100-year floods obtained by each of these methods are summarized in Table 17 below.

Table 17. Summary of Estimated Flows for Standard Frequency Distributions

| Frequency Distribution | Estimated Flow, CFS 10-year | 100-year |
|---|---|---|
| Normal | 15,672 | 23,058 |
| Log-Normal | | |
| Skew, $G_L = 0$ | 13,945 | 35,965 |
| Skew, $G_L = 0.236$ | 14,212 | 42,206 |
| Gumbel | 17,115 | 31,604 |
| Log-Pearson III | | |
| Computed Station Skew, $G_L = 0.236$ | 14,226 | 42,042 |
| WRC Generalized Skew, $\bar{G}_L = -0.252$ | 13,564 | 30,411 |
| WRC Weighted Skew, $G_L = 0.085$ | 14,049 | 38,064 |

There is considerable variation in the 10- and 100-year floods predicted by the general standard frequency distributions. The variation is especially large for the 100-year event where the maximum difference is over 19,000 CFS (510 CMS). The highway designer is faced with the obvious question of which is the appropriate distribution to use for the given set of data.

Considerable insight into the nature of the distribution can be obtained by ordering the flood data, computing the mean, standard deviation and coefficient of skew for the sample and plotting the data on standard probability graph paper. Based on this preliminary graphical analysis, as well as judgement, some standard distributions might be eliminated before the frequency analysis is begun.

Often times, more than one distribution, or in the case of the log-Pearson III, more than one skew option will seem to fit the data fairly well. Some quantitative measure is needed to determine whether one curve or distribution is better than another. Several different techniques have been proposed for this purpose. Two of the most common are the standard error of estimate and confidence limits which are discussed below.

4.3.6.1 Standard Error of Estimate

A common measure of statistical reliability is the standard error of estimate or the root-mean square error. Beard, 1962, gives the standard error of estimate, $S_T$, for the mean, standard deviation and coefficient of skew as

Mean: $$S_T = \frac{S}{\sqrt{n}} \tag{4-29}$$

Standard Deviation: $$S_T = \frac{S}{\sqrt{2n}} \tag{4-30}$$

Coefficient of Skew: $$S_T = \sqrt{\left[6n(n-1)\right] / \left[(n-2)(n+1)(n+3)\right]} \tag{4-31}$$

These equations show that the standard error of estimate is inversely proportional to the square root of the period of record. In other words, the shorter the record, the larger the standard errors. For example, standard errors for a short record will be approximately twice as large as those for a record four times as long.

Kite, 1977, has analyzed standard errors of estimate for flood predictions at various return periods for the normal, log-normal, extreme value, and log-Pearson III standard frequency distributions.

For each of these distributions, the standard error of estimate is given by Kite as

$$S_T = \frac{\delta S}{\sqrt{n}} \tag{4-32}$$

where values of $\delta$ have been calculated from equations given by Kite, 1977. These values are tabulated in Tables 18, 19, 20, and 21 for the normal, log-normal, extreme value and log-Pearson III distributions, respectively. For the normal distribution, $\delta$ is a function of the return period and for the log-normal distributions, $\delta$ is given as a function of the return period and the log coefficient of variation, $(S_L/\overline{Q}_L)$. For the Gumbel distribution, the value of $\delta$ is given in terms of the return period and sample size, while for the log-Pearson III distribution, $\delta$ is given in terms of return period and coefficient of skew.

Standard errors of estimate for the 100-year flood on the Medina River example are computed for the four distributions using Equation (4-32) and Tables 18, 19, 20, and 21.

normal: $S_{100} = \delta \frac{S}{\sqrt{n}} = (1.925)(7074.5) / \sqrt{43} = 2077$ CFS (59 CMS)

log-normal: $S_{100} = \delta \frac{S}{\sqrt{n}} = (2.344)(7074.5) / \sqrt{43} = 2529$ CFS (72 CMS)

Gumbel Extreme Value: $S_{100} = \delta \frac{S}{\sqrt{n}} = (4.331)(7074.5) / \sqrt{43} = 4673$ CFS (132 CMS)

log-Pearson III: ($G_L = 0.085$) $S_{100} = \delta \frac{S}{\sqrt{n}} = (2.790)(7074.5) / \sqrt{43} = 3010$ CFS (85 CMS)

There is also another method for calculating the standard error for the normal distribution. Table 22 from Kite, 1977, gives the ratio of the standard error for a flood with return period, Tr, to the standard deviation of the sample data in terms of the period of record.

Table 18. Parameters $\delta$ for Standard Error of Normal Distribution

| Exceedance Probability in % | | | | | | |
|---|---|---|---|---|---|---|
| 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
| Corresponding Return Period in Years | | | | | | |
| 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 1.0000 | 1.1637 | 1.3496 | 1.5916 | 1.7634 | 1.9253 | 2.2624 |

Table 19. Parameter $\delta$ for Standard Error of Log-Normal Distribution

| Coef. of Var. | Exceedance Probability in % | | | | | | |
|---|---|---|---|---|---|---|---|
| | 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
| | Corresponding Return Period in Years | | | | | | |
| | 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 0.05 | 0.9983 | 1.2162 | 1.4323 | 1.7105 | 1.9087 | 2.0968 | 2.4939 |
| 0.10 | 0.9932 | 1.2698 | 1.5222 | 1.8453 | 2.0766 | 2.2979 | 2.7714 |
| 0.15 | 0.9848 | 1.3241 | 1.6187 | 1.9956 | 2.2676 | 2.5298 | 3.0993 |
| 0.20 | 0.9733 | 1.3784 | 1.7211 | 2.1613 | 2.4819 | 2.7940 | 3.4820 |
| 0.25 | 0.9589 | 1.4323 | 1.8289 | 2.3423 | 2.7202 | 3.0917 | 3.9241 |
| 0.30 | 0.9420 | 1.4855 | 1.9417 | 2.5383 | 2.9829 | 3.4246 | 4.4305 |
| 0.35 | 0.9229 | 1.5378 | 2.0591 | 2.7496 | 3.2708 | 3.7942 | 5.0065 |
| 0.40 | 0.9021 | 1.5890 | 2.1811 | 2.9762 | 3.5845 | 4.2023 | 5.6574 |
| 0.45 | 0.8801 | 1.6389 | 2.3074 | 3.2184 | 3.9251 | 4.6508 | 6.3890 |
| 0.50 | 0.8575 | 1.6876 | 2.4382 | 3.4766 | 4.2935 | 5.1418 | 7.2076 |
| 0.55 | 0.8351 | 1.7351 | 2.5735 | 3.7514 | 4.6910 | 5.6774 | 8.1196 |
| 0.60 | 0.8138 | 1.7814 | 2.7134 | 4.0435 | 5.1190 | 6.2604 | 9.1322 |
| 0.65 | 0.7945 | 1.8266 | 2.8583 | 4.3535 | 5.5790 | 6.8934 | 10.2529 |
| 0.70 | 0.7784 | 1.8709 | 3.0085 | 4.6826 | 6.0729 | 7.5794 | 11.4897 |
| 0.75 | 0.7669 | 1.9143 | 3.1644 | 5.0316 | 6.6024 | 8.3217 | 12.8513 |
| 0.80 | 0.7615 | 1.9570 | 3.3264 | 5.4018 | 7.1698 | 9.1238 | 14.3468 |
| 0.85 | 0.7635 | 1.9991 | 3.4949 | 5.7945 | 7.7773 | 9.9894 | 15.9861 |
| 0.90 | 0.7746 | 2.0408 | 3.6705 | 6.2109 | 8.4272 | 10.9225 | 17.7796 |
| 0.95 | 0.7959 | 2.0821 | 3.8536 | 6.6524 | 9.1221 | 11.9272 | 19.7381 |
| 1.00 | 0.8284 | 2.1232 | 4.0449 | 7.1206 | 9.8646 | 13.0081 | 21.8734 |

Table 20. Parameter δ for Standard Error of Gumbel Extreme Value Distribution

| Sample Size n | Exceedance Probability in % 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
|---|---|---|---|---|---|---|---|
| | Corresponding Return Period in Years 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 10 | 0.9305 | 1.8540 | 2.6200 | 3.6275 | 4.3870 | 5.1460 | 6.9103 |
| 15 | 0.9270 | 1.7695 | 2.4756 | 3.4083 | 4.1127 | 4.8173 | 6.4565 |
| 20 | 0.9250 | 1.7249 | 2.3990 | 3.2919 | 3.9670 | 4.6427 | 6.2154 |
| 25 | 0.9237 | 1.6968 | 2.3507 | 3.2183 | 3.8748 | 4.5322 | 6.0626 |
| 30 | 0.9229 | 1.6772 | 2.3169 | 3.1667 | 3.8103 | 4.4547 | 5.9556 |
| 35 | 0.9223 | 1.6627 | 2.2919 | 3.1286 | 3.7624 | 4.3973 | 5.8763 |
| 40 | 0.9218 | 1.6514 | 2.2725 | 3.0990 | 3.7253 | 4.3528 | 5.8147 |
| 45 | 0.9214 | 1.6424 | 2.2569 | 3.0752 | 3.6955 | 4.3171 | 5.7653 |
| 50 | 0.9211 | 1.6350 | 2.2441 | 3.0555 | 3.6707 | 4.2874 | 5.7242 |
| 55 | 0.9208 | 1.6288 | 2.2333 | 3.0390 | 3.6502 | 4.2626 | 5.6900 |
| 60 | 0.9206 | 1.6235 | 2.2241 | 3.0249 | 3.6325 | 4.2414 | 5.6607 |
| 65 | 0.9204 | 1.6190 | 2.2163 | 3.0130 | 3.6175 | 4.2234 | 5.6357 |
| 70 | 0.9202 | 1.6149 | 2.2092 | 3.0022 | 3.6039 | 4.2071 | 5.6132 |
| 75 | 0.9200 | 1.6114 | 2.2032 | 2.9929 | 3.5923 | 4.1932 | 5.5939 |
| 80 | 0.9199 | 1.6083 | 2.1977 | 2.9846 | 3.5818 | 4.1806 | 5.5765 |
| 85 | 0.9198 | 1.6055 | 2.1929 | 2.9771 | 3.5725 | 4.1694 | 5.5610 |
| 90 | 0.9197 | 1.6030 | 2.1885 | 2.9704 | 3.5640 | 4.1592 | 5.5468 |
| 95 | 0.9196 | 1.6007 | 2.1845 | 2.9643 | 3.5563 | 4.1500 | 5.5341 |
| 100 | 0.9195 | 1.5986 | 2.1808 | 2.9586 | 3.5492 | 4.1414 | 5.5222 |

Table 21. Parameter δ for Standard Error of Log-Pearson Type III Distribution

| Coef. of Skew | Exceedance Probability in % | | | | | | |
|---|---|---|---|---|---|---|---|
| | 50.0 | 20.0 | 10.0 | 4.0 | 2.0 | 1.0 | 0.2 |
| | Corresponding Return Period in Years | | | | | | |
| | 2 | 5 | 10 | 25 | 50 | 100 | 500 |
| 0.0 | 1.0801 | 1.1698 | 1.3748 | 1.8013 | 2.1992 | 2.6369 | 3.7212 |
| 0.1 | 1.0808 | 1.2006 | 1.4368 | 1.9092 | 2.3429 | 2.8174 | 3.9902 |
| 0.2 | 1.0830 | 1.2310 | 1.4990 | 2.0229 | 2.4990 | 3.0181 | 4.3001 |
| 0.3 | 1.0866 | 1.2610 | 1.5611 | 2.1414 | 2.6661 | 3.2373 | 4.6486 |
| 0.4 | 1.0918 | 1.2906 | 1.6228 | 2.2639 | 2.8428 | 3.4732 | 5.0336 |
| 0.5 | 1.0987 | 1.3200 | 1.6840 | 2.3898 | 3.0283 | 3.7247 | 5.4534 |
| 0.6 | 1.1073 | 1.3493 | 1.7442 | 2.5182 | 3.2215 | 3.9905 | 5.9066 |
| 0.7 | 1.1179 | 1.3786 | 1.8033 | 2.6486 | 3.4215 | 4.2695 | 6.3920 |
| 0.8 | 1.1304 | 1.4083 | 1.8611 | 2.7802 | 3.6274 | 4.5607 | 6.9085 |
| 0.9 | 1.1449 | 1.4386 | 1.9172 | 2.9123 | 3.8383 | 4.8631 | 7.4550 |
| 1.0 | 1.1614 | 1.4701 | 1.9717 | 3.0442 | 4.0532 | 5.1756 | 8.0303 |
| 1.1 | 1.1799 | 1.5032 | 2.0243 | 3.1751 | 4.2711 | 5.4969 | 8.6335 |
| 1.2 | 1.2003 | 1.5385 | 2.0751 | 3.3043 | 4.4909 | 5.8259 | 9.2631 |
| 1.3 | 1.2223 | 1.5767 | 2.1242 | 3.4311 | 4.7115 | 6.1613 | 9.9177 |
| 1.4 | 1.2457 | 1.6186 | 2.1718 | 3.5546 | 4.9319 | 6.5017 | 10.5958 |
| 1.5 | 1.2701 | 1.6649 | 2.2182 | 3.6741 | 5.1507 | 6.8456 | 11.2957 |
| 1.6 | 1.2951 | 1.7164 | 2.2640 | 3.7891 | 5.3669 | 7.1915 | 12.0155 |
| 1.7 | 1.3202 | 1.7741 | 2.3097 | 3.8989 | 5.5792 | 7.5378 | 12.7531 |
| 1.8 | 1.3450 | 1.8385 | 2.3562 | 4.0029 | 5.7865 | 7.8829 | 13.5064 |
| 1.9 | 1.3687 | 1.9104 | 2.4046 | 4.1008 | 5.9875 | 8.2252 | 14.2731 |
| 2.0 | 1.3907 | 1.9904 | 2.4560 | 4.1922 | 6.1812 | 8.5629 | 15.0508 |

Table 22. Dimensionless Ratio of the Standard Error of the T-Year Event to the Standard Deviation of the Annual Events for Normal and Log Normal Distributions

| Return Period Tr | Sample Length, n | | | | | |
|---|---|---|---|---|---|---|
| | 2 | 5 | 10 | 20 | 50 | 100 |
| 2 | 0.707 | 0.447 | 0.316 | 0.224 | 0.141 | 0.100 |
| 5 | 0.782 | 0.495 | 0.350 | 0.247 | 0.156 | 0.116 |
| 10 | 0.954 | 0.604 | 0.427 | 0.302 | 0.191 | 0.135 |
| 20 | 1.083 | 0.685 | 0.484 | 0.342 | 0.217 | 0.153 |
| 50 | 1.208 | 0.764 | 0.540 | 0.382 | 0.242 | 0.176 |
| 100 | 1.364 | 0.863 | 0.610 | 0.431 | 0.273 | 0.193 |

from Kite, 1977

The standard error of estimate for the 100-year flood on the Medina River data is calculated below:

Sample length = 43 Years

Return Period, Tr = 100 Years

From Table 22, Ratio = 0.31

$$S_{100} = (.31)\ S = (.31)(7074.5) = 2193 \text{ CFS } (62 \text{ CMS})$$

This is very close to the standard error calculated with Equation (4-32) which was 2077 CFS (59 CMS).

The standard error computed in this manner is actually a measure of the variance that could be expected in a predicted T-year event if the event were estimated from each of a very large number of equally good samples of equal length. Because of its critical dependence on the period of record, the standard error is difficult to interpret, and a large value may be the reflection of a short record. For example, the standard error for the log-Pearson III estimate of the 100-year flood is relatively large. However, the 43-year period of record is statistically of insufficient length to properly evaluate the station skew, and the potential variability in the prediction of the 100-year flood is shown by the standard error of estimate. For this reason, some hydrologists prefer confidence limits for evaluating the reliability of a selected frequency distribution.

### 4.3.6.2 Confidence Limits

Confidence limits are used to estimate the uncertainties associated with the determination of floods of specified return periods from frequency distributions. Since a given frequency distribution is only an estimated determinant from a sample of a population, it is probable that another sample from the same stream of equal length but taken at a different time would yield a different frequency curve. Confidence limits, or more correctly, confidence intervals, define the range within which these frequency curves could be expected to fall with specified confidence or levels of significance.

Bulletin 17B outlines a method for developing upper and lower confidence intervals. The general forms of the confidence limit equations are

$$U_{P,C}\,(Q) = \bar{Q} + SK^{U}_{P,C} \tag{4-33}$$

and

$$L_{P,C}\,(Q) = \bar{Q} + SK^{L}_{P,C} \tag{4-34}$$

where $U_{p,c}(Q)$ and $L_{p,c}(Q)$ are the upper and lower confidence limits for a flow, Q, at a level of confidence, c, and exceedance probability, p; and $K^{U}_{p,c}$ and $K^{L}_{p,c}$ are the upper and lower confidence coefficients at the specified values of p and c. Values of $K^{U}_{p,c}$ and $K^{L}_{p,c}$ for normal distribution are given in Table 23 for the commonly used confidence levels of 0.05 and 0.95. Bulletin 17B, from which Table 23 was abstracted, contains a more extensive table covering other confidence levels.

Confidence limits defined in this manner are called one-sided because each defines the limit on just one side of the frequency curve. The one-sided intervals can be combined to form a two-sided confidence limit such that the combination of 95 percent and 5 percent confidence limits define a 90 percent confidence limit. Practically, this means that at a specified exceedance probability or return period, there is a 5 percent chance the flow will exceed the upper confidence limit value and a 5 percent chance the flow will be less than the lower confidence limit value. Stated another way, it can be expected that 90 percent of the time, the specified frequency flow will fall within the two confidence limits.

When the skew is non-zero, Bulletin 17B gives the following approximate equations for estimating values of $K^{U}_{p,c}$ and $K^{L}_{p,c}$ in terms of the value of $K_{G,p}$ for the given skew and exceedance probability

Table 23. Confidence Limit Deviate Values for Normal and Log-Normal Distributions

| Confidence Level | Systematic Record Length n | Exceedance Probability .002 | .010 | .020 | .040 | .100 | .200 | .500 | .800 | .990 |
|---|---|---|---|---|---|---|---|---|---|---|
| .05 | 10 | 4.862 | 3.981 | 3.549 | 3.075 | 2.355 | 1.702 | .580 | -.317 | -1.563 |
| | 15 | 4.304 | 3.520 | 3.136 | 2.713 | 2.068 | 1.482 | .455 | -.406 | -1.677 |
| | 20 | 4.033 | 3.295 | 2.934 | 2.534 | 1.926 | 1.370 | .387 | -.460 | -1.749 |
| | 25 | 3.868 | 3.158 | 2.809 | 2.425 | 1.838 | 1.301 | .342 | -.497 | -1.801 |
| | 30 | 3.755 | 3.064 | 2.724 | 2.350 | 1.777 | 1.252 | .310 | -.525 | -1.840 |
| | 40 | 3.608 | 2.941 | 2.613 | 2.251 | 1.697 | 1.188 | .266 | -.656 | -1.896 |
| | 50 | 3.515 | 2.862 | 2.542 | 2.188 | 1.646 | 1.146 | .237 | -.592 | -1.936 |
| | 60 | 3.448 | 2.807 | 2.492 | 2.143 | 1.609 | 1.116 | .216 | -.612 | -1.966 |
| | 70 | 3.399 | 2.765 | 2.454 | 2.110 | 1.581 | 1.093 | .199 | -.629 | -1.990 |
| | 80 | 3.360 | 2.733 | 2.425 | 2.083 | 1.559 | 1.076 | .186 | -.642 | -2.010 |
| | 90 | 3.328 | 2.706 | 2.400 | 2.062 | 1.542 | 1.061 | .175 | -.652 | -2.026 |
| | 100 | 3.301 | 2.684 | 2.380 | 2.044 | 1.527 | 1.049 | .166 | -.662 | -2.040 |
| .95 | 10 | 1.989 | 1.563 | 1.348 | 1.104 | .712 | .317 | -.580 | -1.702 | -3.981 |
| | 15 | 2.121 | 1.677 | 1.454 | 1.203 | .802 | .406 | -.455 | -1.482 | -3.520 |
| | 20 | 2.204 | 1.749 | 1.522 | 1.266 | .858 | .460 | -.387 | -1.370 | -3.295 |
| | 25 | 2.264 | 1.801 | 1.569 | 1.309 | .898 | .497 | -.342 | -1.301 | -3.158 |
| | 30 | 2.310 | 1.840 | 1.605 | 1.342 | .928 | .525 | -.310 | -1.252 | -3.064 |
| | 40 | 2.375 | 1.896 | 1.657 | 1.391 | .970 | .565 | -.266 | -1.188 | -2.941 |
| | 50 | 2.421 | 1.936 | 1.694 | 1.424 | 1.000 | .592 | -.237 | -1.146 | -2.862 |
| | 60 | 2.456 | 1.966 | 1.722 | 1.450 | 1.022 | .612 | -.216 | -1.116 | -2.807 |
| | 70 | 2.484 | 1.990 | 1.745 | 1.470 | 1.040 | .629 | -.199 | -1.093 | -2.765 |
| | 80 | 2.507 | 2.010 | 1.762 | 1.487 | 1.054 | .642 | -.186 | -1.076 | -2.733 |
| | 90 | 2.526 | 2.026 | 1.778 | 1.500 | 1.066 | .652 | -.175 | -1.061 | -2.706 |
| | 100 | 2.542 | 2.040 | 1.791 | 1.512 | 1.077 | .662 | -.166 | -1.049 | -2.684 |

from WRC, 1981

$$K^{U}_{P,c} = \frac{K_{G,P} + \sqrt{K^2_{G,P} - ab}}{a} \qquad (4\text{-}35)$$

and

$$K^{L}_{P,c} = \frac{K_{G,P} - \sqrt{K^2_{G,P} - ab}}{a} \qquad (4\text{-}36)$$

where

$$a = 1 - \frac{Z_c^2}{2(n-1)}$$

$$b = K^2_{G,P} - \frac{Z_c^2}{n}$$

and where $Z_c$ is the standard normal deviate (zero-skew Pearson Type III deviate) with exceedance probability of (1-c).

For the Gumbel extreme value distribution, Kite, 1977, gives the upper and lower 95 percent confidence limits as

$$Q_T \pm 1.96\, S_T \qquad (4\text{-}37)$$

where $S_T$ is determined from Equation (4-32), and Table 20.

Confidence limits for each of the standard distributions have been computed in accordance with the above discussion. These are illustrated in Figures 31, 32, 33 and 34 which show the standard frequency curve and confidence intervals at the 0.05 and 0.95 level of significance. Although the methods are not consistent with one another, the confidence limit curves give comparable results.

Based on the computed confidence limits, it appears that a log-Pearson III would be the most acceptable distribution for the Medina River data. The actual data follow the distribution very well, and all the data fall within the confidence intervals. Compared to the log-normal distribution which also provides a reasonable fit, it is to be noted that the confidence limits for the log-Pearson III distribution are a little narrower or tighter at the upper and lower ends of the curve. Based on this analysis, the log-Pearson Type III would be the preferred standard distribution with log-normal also acceptable. The normal and Gumbel distributions are unsatisfactory for this particular set of data.

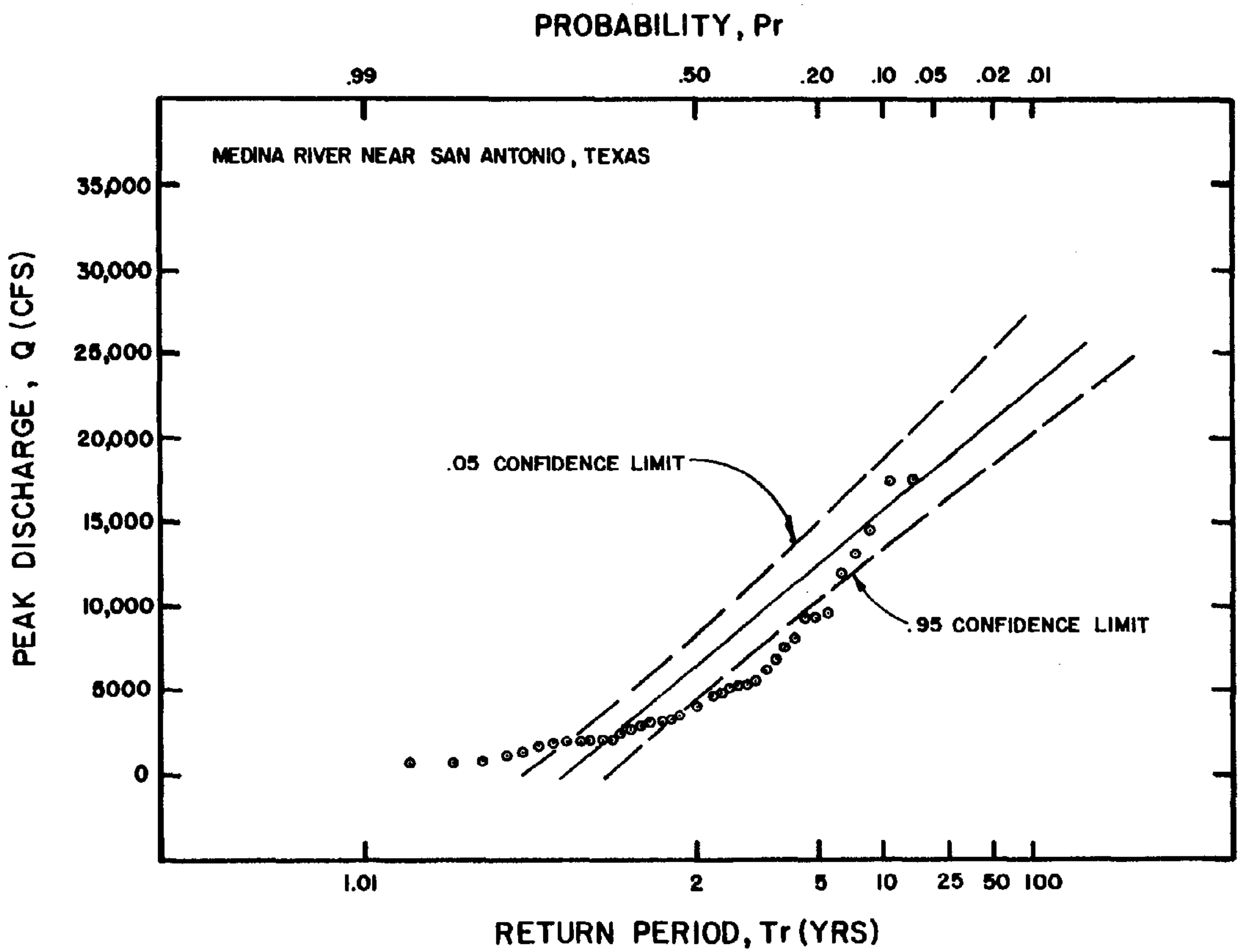



Figure 31. Normal Distribution with Confidence Limits, Medina River, TX

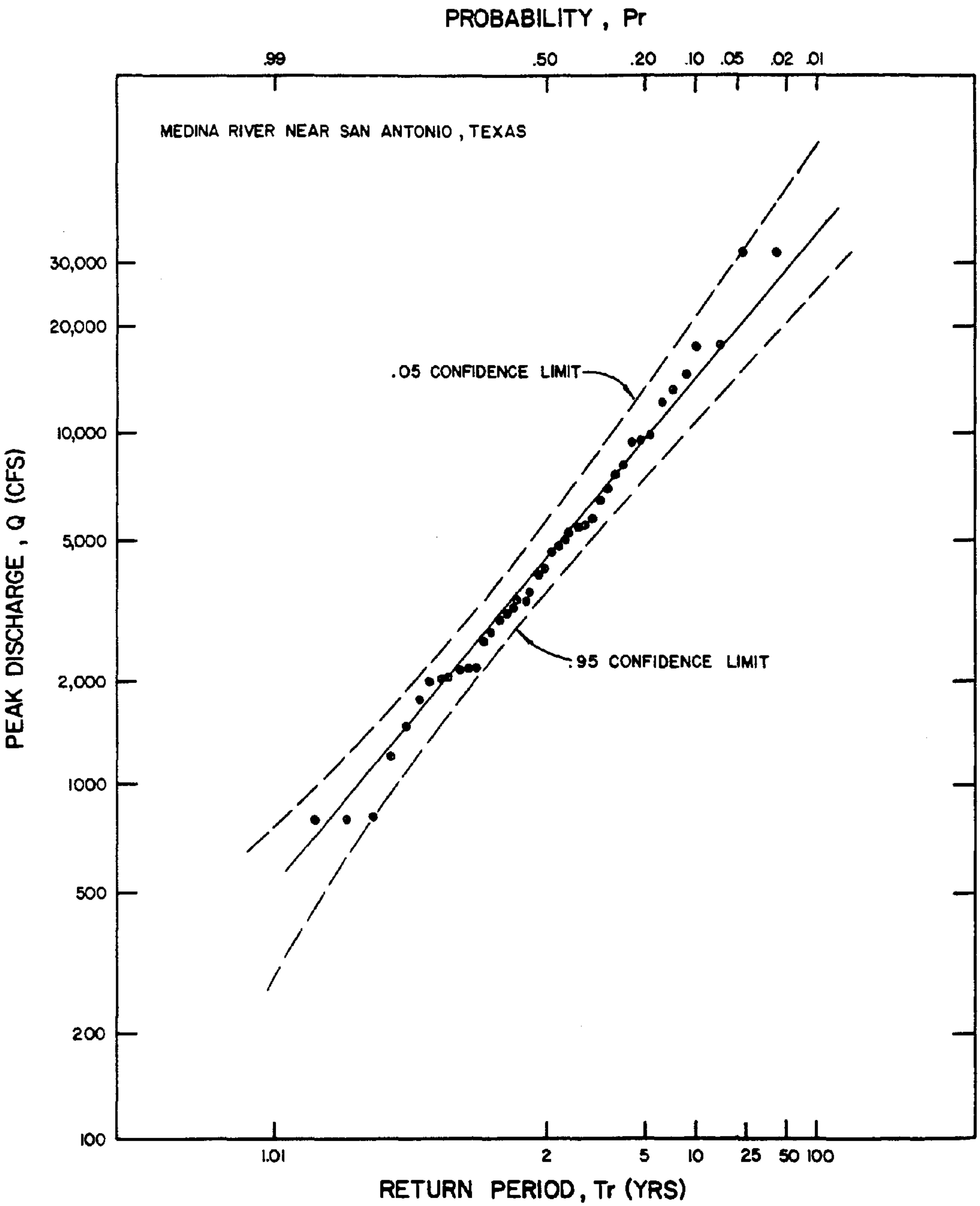



Figure 32. Log-Normal Distribution with Confidence Limits, Medina River, Texas

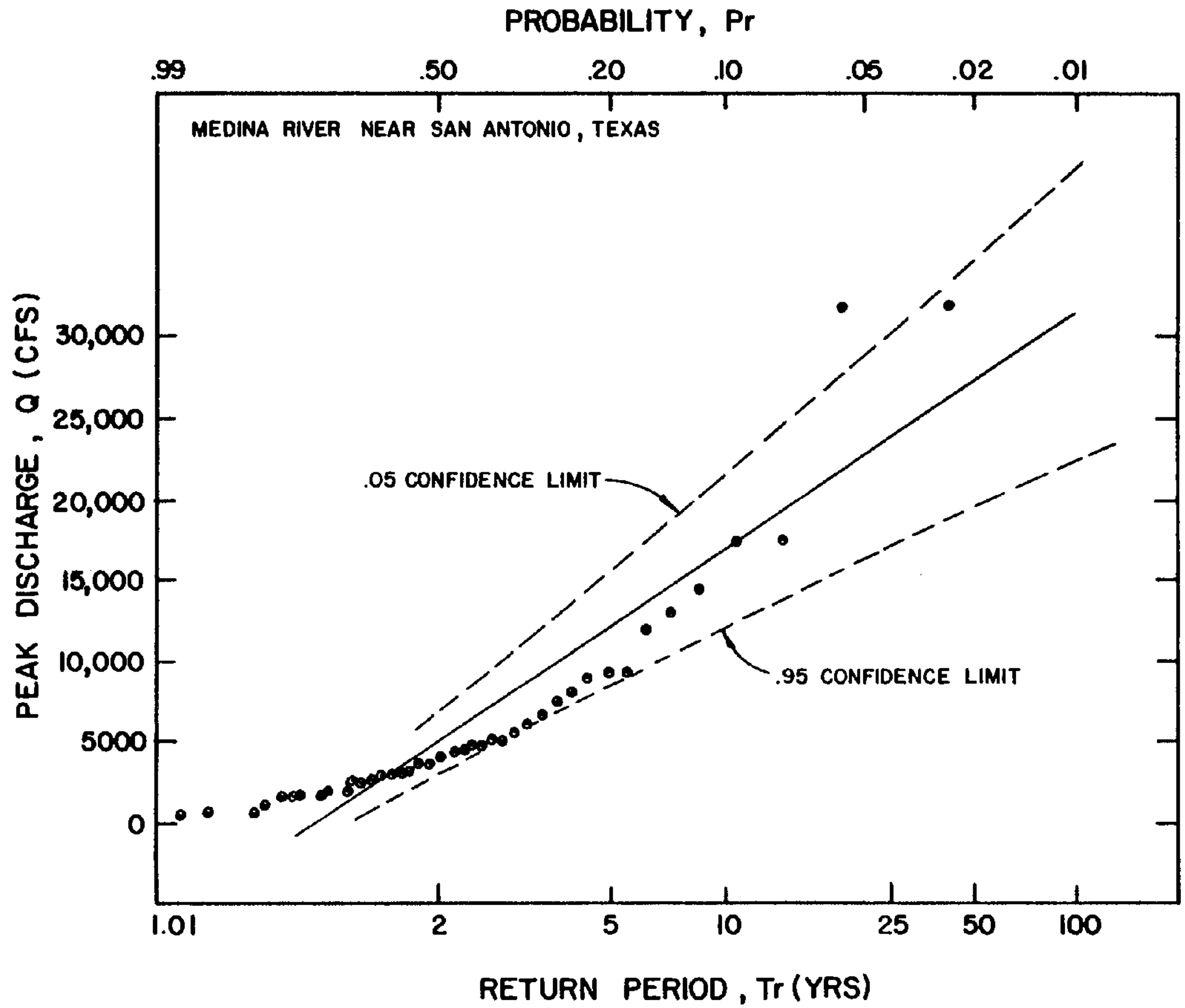


Figure 33. Gumbel Extreme Value Distribution with Confidence Limits, Medina River, TX

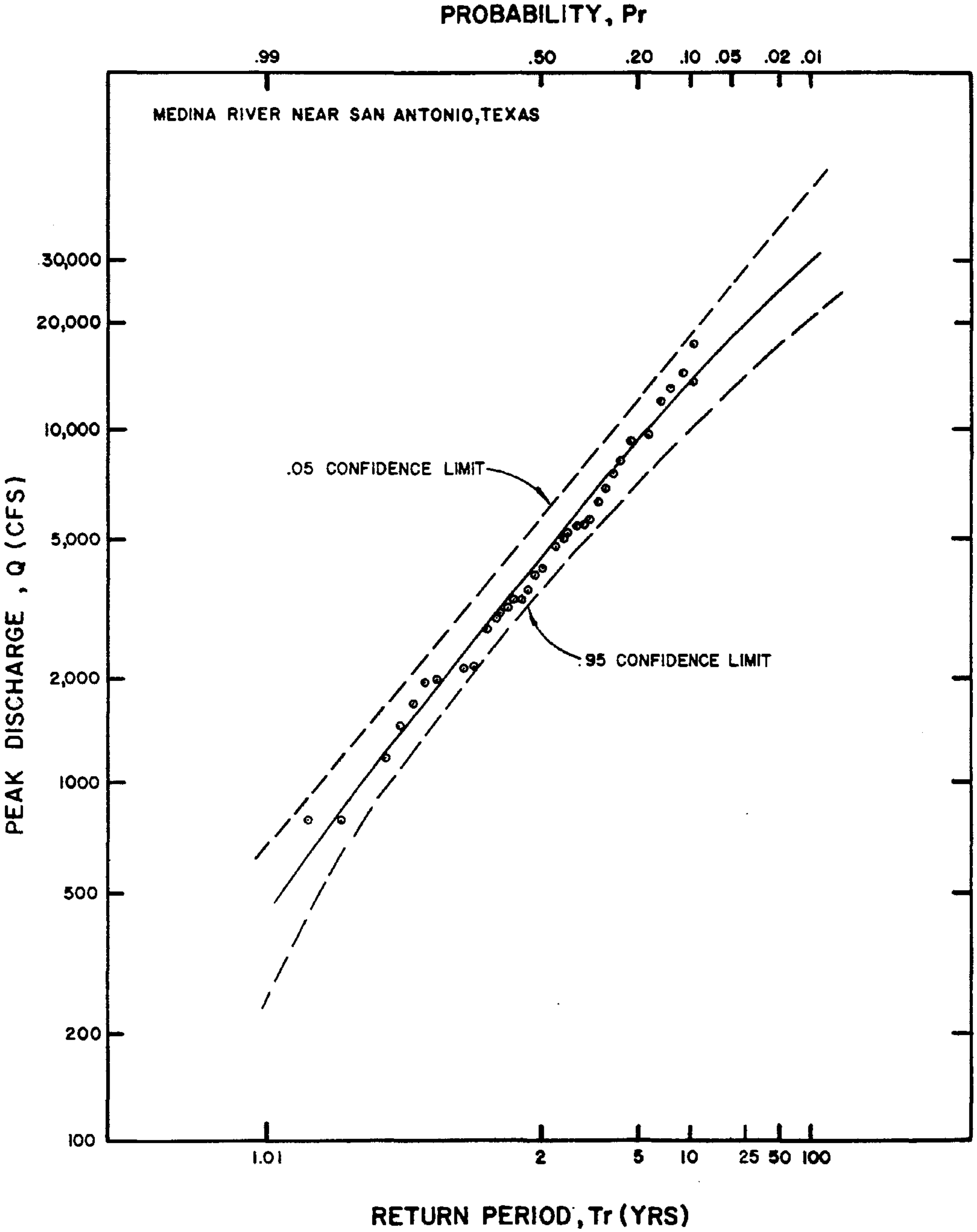



Figure 34. Log Pearson Type III Distribution with Confidence Limits, Medina River, Texas

### 4.3.7 Other Data Considerations in Frequency Analysis

In the course of performing frequency analyses for various watersheds, the designer will undoubtedly encounter situations where further adjustments to the data are indicated. Additional analysis may be necessary due to outliers, inclusion of historical data, incomplete records or years with zero flow and mixed populations. Some of the more common methods of analysis are discussed in the following paragraphs.

#### 4.3.7.1 Outliers

Outliers, which may be found at either or both ends of a frequency distribution, are data points that occur, but appear to belong to a sample of a different size. This is reflected in one or more data points not following the trend of the remaining data.

Bulletin 17B presents criteria based on a one-sided test to detect outliers at a 10 percent significance level. If the station skew is greater than 0.4, tests are applied for high outliers first; and if less than -0.4, low outliers are considered first. If the station skew is between $\pm$ 0.4, both high and low outliers are tested before any data are eliminated. The detection of high and low outliers is obtained with the equations

High Outlier: $$Q_L = \bar{Q}_L - K_N S_L \qquad (4\text{-}38a)$$

and

Low Outlier: $$Q_L = \bar{Q}_L + K_N S_L \qquad (4\text{-}38b)$$

where $Q_L$ is the log of the high or low outlier limit, $\bar{Q}_L$ is the mean of the log of the sample flows, $S_L$ is the standard deviation of the sample of $Q_L$, and $K_N$ is the critical deviate taken from Table 24.

To illustrate, this criteria for outlier detection, Equations (4-38a) and (4-38b) are applied to the 43-year record for the Medina River which has $\bar{Q}_L$ = 3.639 and $S_L$ = 0.394. From Table 24, $K_n$ = 2.710. Testing first for high outliers,

$$Q_L = 3.639 + 2.710\,(0.394) = 4.707$$

$$Q = 10^{(4.707)} = 50{,}933 \text{ CFS } (1442 \text{ CMS})$$

Table 24. Outlier Test K Values at 10 Percent Significance Level

| Sample size | K value | Sample size | K value | Sample size | K value | Sample size | K value |
|---|---|---|---|---|---|---|---|
| 10 | 2.036 | 45 | 2.727 | 80 | 2.940 | 115 | 3.064 |
| 11 | 2.088 | 46 | 2.736 | 81 | 2.945 | 116 | 3.067 |
| 12 | 2.134 | 47 | 2.744 | 82 | 2.949 | 117 | 3.070 |
| 13 | 2.165 | 48 | 2.753 | 83 | 2.953 | 118 | 3.073 |
| 14 | 2.213 | 49 | 2.760 | 84 | 2.957 | 119 | 3.075 |
| 15 | 2.247 | 50 | 2.768 | 85 | 2.961 | 120 | 3.078 |
| 16 | 2.279 | 51 | 2.775 | 86 | 2.966 | 121 | 3.081 |
| 17 | 2.309 | 52 | 2.783 | 87 | 2.970 | 122 | 3.083 |
| 18 | 2.335 | 53 | 2.790 | 88 | 2.973 | 123 | 3.086 |
| 19 | 2.361 | 54 | 2.798 | 89 | 2.977 | 124 | 3.089 |
| 20 | 2.385 | 55 | 2.804 | 90 | 2.981 | 125 | 3.092 |
| 21 | 2.408 | 56 | 2.811 | 91 | 2.984 | 126 | 3.095 |
| 22 | 2.429 | 57 | 2.818 | 92 | 2.989 | 127 | 3.097 |
| 23 | 2.448 | 58 | 2.824 | 93 | 2.993 | 128 | 3.100 |
| 24 | 2.467 | 59 | 2.831 | 94 | 2.996 | 129 | 3.102 |
| 25 | 2.487 | 60 | 2.837 | 95 | 3.000 | 130 | 3.104 |
| 26 | 2.502 | 61 | 2.842 | 96 | 3.003 | 131 | 3.107 |
| 27 | 2.510 | 62 | 2.849 | 97 | 3.006 | 132 | 3.109 |
| 28 | 2.534 | 63 | 2.854 | 98 | 3.011 | 133 | 3.112 |
| 29 | 2.549 | 64 | 2.860 | 99 | 3.014 | 134 | 3.114 |
| 30 | 2.563 | 65 | 2.866 | 100 | 3.017 | 135 | 3.116 |
| 31 | 2.577 | 76 | 2.871 | 101 | 3.021 | 136 | 3.119 |
| 32 | 2.591 | 67 | 2.877 | 102 | 3.024 | 137 | 3.122 |
| 33 | 2.604 | 68 | 2.883 | 103 | 3.027 | 138 | 3.124 |
| 34 | 2.616 | 69 | 2.888 | 104 | 3.030 | 139 | 3.126 |
| 35 | 2.628 | 70 | 2.893 | 105 | 3.033 | 140 | 3.129 |
| 36 | 2.639 | 71 | 2.897 | 106 | 3.037 | 141 | 3.131 |
| 37 | 2.650 | 72 | 2.903 | 107 | 3.040 | 142 | 3.133 |
| 38 | 2.661 | 73 | 2.908 | 108 | 3.043 | 143 | 3.135 |
| 39 | 2.671 | 74 | 2.912 | 109 | 3.046 | 144 | 3.138 |
| 40 | 2.682 | 75 | 2.917 | 110 | 3.049 | 145 | 3.140 |
| 41 | 2.692 | 76 | 2.922 | 111 | 3.052 | 146 | 3.142 |
| 42 | 2.700 | 77 | 2.927 | 112 | 3.055 | 147 | 3.144 |
| 43 | 2.710 | 78 | 2.931 | 113 | 3.058 | 148 | 3.146 |
| 44 | 2.710 | 79 | 2.935 | 114 | 3.061 | 149 | 3.148 |

from WRC, 1981

No flows in the sample exceed this amount, so there are no high outliers. Now testing for low outliers, Equation (4-38b)

$$Q_L = 3.639 - 2.710\ (0.394) = 2.571$$

$$Q = 10^{(2.571)} = 372\ \text{CFS}\ (11\ \text{CMS})$$

There are no flows in the Medina River sample that are less than this critical value. Therefore, the entire sample is used in this log-Pearson III analysis.

If the sample is found to contain high outliers, the peak flows should be checked against historical data and data from nearby stations before discarding the data from the sample. If a high outlier is adjusted based on historical data, the mean and standard deviation of the log distribution should be recomputed for the adjusted data before testing for low outliers.

The SCS National Engineering Handbook, 1972, presents a similar procedure for testing for high and low outliers based on Five-Percent Two-Sided Critical Deviates for a normal distribution. The detection criteria is identical to that used for the log-Pearson III method described above except that the value of $K_N$ is taken from an appropriate table contained in the SCS Handbook for values of the critical deviate. The SCS procedure involves an iterative procedure wherein the sample characteristics are used to test for successive outliers. If the first data point is determined to be an outlier and discarded, new sample chacteristics are determined and the next data point is tested. The procedure is repeated until no further outliers are detected.

Regardless of the technique used to test for outliers, the designer should consider the possibility of other standard distributions if more than one or two outliers are detected. If a better distribution can be found, it should be used and again tested for outliers. If a better distribution cannot be found, the designer may then either adjust the outliers for historical data in the case of high outliers, treat the low outliers as missing data, or simply keep or eliminate the data from the sample. This latter decision is judgmental and will depend on the use of the frequency analysis and the designer's experience and understanding of the hydrologic and physical characteristics of the watershed.

4.3.7.2 Historical Data

When there is reliable information indicating that one or more large floods occurred outside the period of record, the frequency analysis should be adjusted to account for these events. Although estimates of unrecorded historical flood discharges may be inaccurate, they should be incorporated into the sample because the error in estimating the flow is small in relation to the chance variability in the peak flows from year to year. If, however, there is evidence these floods resulted under different watershed conditions or from situations that differ from the sample, the large floods should be rejected as outliers or some other analysis used.

Bulletin 17B provides methods to adjust for historical data based on the assumption that "the data from the systematic (station) record is representative of the intervening period between the systematic and historic record lengths." Two sets of equations for this adjustment are given in Bulletin 17B. The first is applied directly to the log-transformed station data including the historical events. The floods are reordered, assigning the largest historic flood a rank of one. The order number is then weighted giving a weighting of 1.00 to the historic event, and weighting the station data order by a value determined from the equation

$$W = \frac{H - Z}{n + L} \tag{4-39}$$

where W is the weighting factor, H is the historically longer period of years, Z is the number of historical events included in the analysis and L is the number of low outliers excluded from the analysis. The properties of the historically extended sample are then computed according to the equations

$$\bar{Q}'_L = \frac{W \Sigma Q_L + \Sigma Q_{L,Z}}{H - WL} \tag{4-40}$$

$$S'^2_L = \frac{W \Sigma (Q_L - \bar{Q}'_L)^2 + \Sigma (Q_{L,Z} - \bar{Q}'_L)^2}{H - WL - 1} \tag{4-41}$$

and

$$G'_L = \frac{H - WL}{(H - WL - 1)(H - WL - 2)} \left[ \frac{W \Sigma (Q_L - \bar{Q}'_L)^3 + \Sigma (Q_{L,Z} - \bar{Q}'_L)^3}{S'^3_L} \right] \tag{4-42}$$

where $\bar{Q}'_L$ is the historically adjusted mean log transform of the flows, $Q_L$ is the log transform of the flows contained in the sample record, $Q_{L,Z}$ is the log of the historic peak flow, $S'_L$ is the historically adjusted standard deviation and $G'_L$ is the historically adjusted skew coefficient. All other values are as previously defined.

In the case where the sample properties were previously computed such as were done for the Medina River in Table 12, Bulletin 17B gives the following adjustments which can be applied directly

$$\bar{Q}'_L = \frac{Wn\,\bar{Q}_L + \Sigma Q_{L,Z}}{H-WL} \qquad (4\text{-}43)$$

$$S_L'^2 = \frac{W(n-1)\,S_L^2 + Wn(\bar{Q}_L - \bar{Q}'_L)^2 + \Sigma(Q_{L,Z} - \bar{Q}'_L)^2}{(H-WL-1)} \qquad (4\text{-}44)$$

and

$$G'_L = \frac{H-WL}{(H-WL-1)(H-WL-2)\,S_L'^3}\left[\frac{W(n-1)(n-2)\,S_L^3\,G_L}{n} + 3W(n-1)(\bar{Q}_L - \bar{Q}'_L)\,S_L^2 + Wn(\bar{Q}_L - \bar{Q}'_L)^3 + \Sigma(Q_{L,Z} - \bar{Q}'_L)^3\right] \qquad (4\text{-}45)$$

Once the adjusted statistical parameters are determined, the log-Pearson III distribution is determined by Equation (4-26) using a plotting position determined by the Weibull formula

$$P = \frac{m'}{H+1} \qquad (4\text{-}46)$$

where m' is the adjusted order number of the floods including historical events, where

$m' = m$ for $1 \le m \le Z$

$m' = Wm - (W - 1)(Z + 0.5)$ for $(Z + 1) \le m \le (Z + nL)$

Detailed examples illustrating the computations for the historic adjustment are contained in Bulletin 17B and the designer is referred to this reference for further information.

4.3.7.3 Incomplete Records and Zero Flows

Streamflow records are often interrupted for a variety of reasons. Gages may be removed for some period of time, there may be periods of zero flow which are common in the arid regions of the United States, and there may be periods when a gage is inoperative either because the flow is too low to record or it is too large and causes a gage malfunction.

If the break in the record is not flood related such as the removal of a gage, no special adjustments are needed and the segments of the interrupted record can be combined together to produce a record equal to the sum of the length of the segments. When a gage malfunctions during a flood, it is usually possible to estimate the peak discharge from highwater marks or slope-area calculations. The estimate is made a part of the record and a frequency analysis performed without adjustment.

Zero flows or flows that are too low to be recorded present more of a problem since in the log transform, these flows produce undefined values. In this case, Bulletin 17B presents an adjustment based on conditional probability which is applicable if not more than 25 percent of the sample is eliminated. The adjustment for zero flows also is applied only after all other data adjustments have been made. The adjustment is made by first calculating the relative frequency, Pa, that the annual peak will exceed the level below which flows are zero, or not considered (the truncation level). In other words,

$$Pa = \frac{M}{n} \qquad (4\text{-}47)$$

where M is the number of flows above the truncated level and n is the total period of record. The exceedance probabilities, P, of selected points on the frequency curve are recomputed as a conditional probability as follows

$$P = Pa \cdot Pd \qquad (4\text{-}48)$$

where Pd is the selected probability. Since the frequency curve adjusted by Equation (4-48) has unknown statistics, its properties, synthetic values, are computed by the equations

$$\bar{Q}_s = \log(Q_{.50}) - K_{.50}(S_s) \qquad (4\text{-}49)$$

$$S_s = \frac{\log(Q_{.01}/Q_{.50})}{K_{.01} - K_{.50}} \qquad (4\text{-}50)$$

and

$$G_s = -2.50 + 3.12\,\frac{\log(Q_{.01}/Q_{.10})}{\log(Q_{.10}/Q_{.50})} \qquad (4\text{-}51)$$

where $\bar{Q}_s$, $S_s$ and $G_s$ are the mean, standard deviation and skew of the synthetic frequency curve, $Q_{.01}$, $Q_{.10}$ and $Q_{.50}$ are discharges with exceedance probabilities of 0.01, 0.10 and 0.50 respectively, and $K_{.01}$ and $K_{.50}$ are the standard log-Pearson III deviates for exceedance probabilities of 0.01 and 0.50 respectively. The values of $Q_{.01}$, $Q_{.10}$ and $Q_{.50}$ must usually be interpolated since probabilites computed with Equation (4-47) are not normally those needed to compute the properties of the synthetic or truncated distribution.

The log-Pearson III distribution can then be computed in the conventional manner using the synthetic statistical properties. Bulletin 17B recommends the distribution be compared with the observed flows since data adjusted for conditional probability may not follow a log-Pearson III distribution.

4.3.7.4 Mixed Populations

In some areas of the United States, floods are caused by combinations of events, e.g., rainfall and snowmelt in mountainous areas or rainfall and hurricane events along the Gulf and Atlantic coasts. Records from such combined events are said to be mixed populations. These records are often characterized by very large skew coefficients and when plotted suggest that two different distributions might be applicable.

Such records should be divided into two separate records according to their respective causes. Each record is analyzed separately by an appropriate frequency distribution. The two separate frequency curves can then be combined through the concept of addition of the probabilities of two non-independent events, Equation (4-52), as follows:

$$Pr\left\{Q \text{ or } Q_m\right\} = Pr\left\{Q\right\} + Pr\left\{Q_m\right\} - \left[Pr\left\{Q\right\} Pr\left\{Q_m\right\}\right] \qquad (4\text{-}52)$$

4.3.7.5 Transposition of Records

In some cases, it is possible to extend and to improve peak flow estimates obtained from short records utilizing longer records from nearby gaged watersheds. Basically, the longer record is used to estimate new statistical parameters for the short record depending on the correlation between the two concurrent records. While individual events can be estimated for the short records by correlation and other methods, Beard, 1962, notes that such methods tend to reduce the variance of the estimated values. Beard then outlines a procedure to extend a short record as follows.

One or more base stations with long records are selected from the same region in order to extend the record at a station with a short record. In order to estimate the degree of correlation between the corresponding flows at the base station and the short record station, the flows and corresponding logarithms for the two stations are arranged chronologically (not by magnitude) for the concurrent periods of record. The mean and standard deviations for the two stations are calculated by Equations (4-11) and (4-12). Also the mean and standard deviation are computed for the base station's period of record that is concurrent with the short-period record. The correlation between the stations is then computed by the equations

$$\bar{R}^2 = 1 - (1 - R^2)\frac{n-1}{n-2} \tag{4-53}$$

and

$$R^2 = \frac{\left[\Sigma Q_L Q_L'' - \Sigma Q_L \Sigma (Q_L''/n)\right]^2}{\left[\Sigma Q_L^2 - (\Sigma Q_L)^2/n\right]\left[\Sigma Q_L''^2 - (\Sigma Q_L'')^2/n\right]} \tag{4-54}$$

where $\bar{R}$ is the adjusted correlation coefficient, $R$ is the unadjusted correlation, $Q_L$ is the logarithm of the peak flow at the short record station and $Q''_L$ is the logarithm of the peak flow at the base station over the concurrent records for n years. The mean and standard deviation of the short period of record are then adjusted for the extended record by the approximate relations

$$\bar{Q}_1' = \bar{Q}_1 + (\bar{Q}_2' - \bar{Q}_2)\, R \frac{S_1}{S_2} \tag{4-55}$$

and

$$S_1' = S_1 + (S_2' - S_2)\, R^2 \frac{S_1}{S_2} \tag{4-56}$$

where the primed values are the mean and standard deviation computed from or adjusted to the long period, the unprimed values are for the short period of record and the subscripts 1 and 2 refer to the gage with the short record and the base record, respectively.

Beard, 1962, then expresses the reliability of the adjusted values in terms of the equivalent length of record necessary to establish equally reliable unadjusted values. The equivalent record is given by

$$n'_1 = \frac{n_1}{1 - \left[(n'_2 - n_1)/n'_2\right]\bar{R}^2} \qquad (4\text{-}57)$$

Thus, the use of another record $(n'_2 - n_1)$ years longer than the short period record n, is equivalent of adding $(n'_1 - n_1)$ years to the short record at the computed adjusted correlation coefficient.

The adjusted frequency distribution is computed in the conventional manner using the adjusted distribution properties, Q', S', and G. The following is an illustrative example from Beard for using a nearby gage record to adjust a shorter record. Given the annual series for two stations as shown in Table 25, it is desired to extend the 30-year record using data from the gage with 47 years of record (the base station).

The means and standard deviations are computed respectively with Equation (4-11) and (4-12) to be 3.666 and 0.303 for the short period station, 4.269 and 0.357 for the base station, and 4.289 and 0.397 for the portion of the base station record that is concurrent with the short period station. The correlation coefficients, $R^2$ and $\bar{R}^2$ can then be computed from the data in Table 25 as follows.

From Equation (4-54)

$$R^2 = \frac{(474.5925 - 471.6980)^2}{(405.7813 - 403.1134)(556.5276 - 551.9514)} = 0.685$$

and from Equation (4-53)

$$\bar{R}^2 = 1 - (1 - 0.685)\frac{29}{28} = 0.674$$

The mean and standard deviation for the short record station can then be adjusted for the base record by Equation (4-55) and (4-56) as follows:

$$\bar{Q}'_1 = 3.666 + (4.269 - 4.289)(0.685)^{0.5}\left(\frac{0.303}{0.397}\right) = 3.653$$

and

$$S'_1 = 0.303 + (0.357 - 0.397)(0.685)\left(\frac{0.303}{0.397}\right) = 0.282$$

Table 25. Annual Series for Transposition of Base Station Record to a Short Record Station

| Year | Short Record Station | | Base Station | |
|---|---|---|---|---|
| | $Q_1$ CFS | Log $Q_1$ | $Q_2$ CFS | Log $Q_2$ |
| 1912 | | | 4,570 | 3.66 |
| 1913 | | | 7,760 | 3.89 |
| 1914 | | | 32,400 | 4.51 |
| 1915 | | | 27,500 | 4.44 |
| 1916 | | | 19,000 | 4.28 |
| 1917 | | | 24,000 | 4.38 |
| 1918 | | | 13,200 | 4.12 |
| 1919 | | | 15,500 | 4.19 |
| 1920 | | | 10,200 | 4.01 |
| 1921 | | | 14,100 | 4.15 |
| 1922 | | | 14,800 | 4.17 |
| 1923 | | | 10,500 | 4.02 |
| 1924 | | | 11,500 | 4.06 |
| 1925 | | | 27,500 | 4.44 |
| 1926 | | | 17,800 | 4.25 |
| 1927 | | | 36,300 | 4.56 |
| 1928 | | | 67,600 | 4.83 |
| 1929 | 1,520 | 3.18 | 5,500 | 3.74 |
| 1930 | 6,000 | 3.78 | 25,500 | 4.41 |
| 1931 | 1,500 | 3.18 | 5,570 | 3.75 |
| 1932 | 5,440 | 3.74 | 9,980 | 4.00 |
| 1933 | 1,080 | 3.03 | 5,100 | 3.71 |
| 1934 | 2,630 | 3.42 | 11,100 | 4.05 |
| 1935 | 4,010 | 3.60 | 25,500 | 4.41 |
| 1936 | 4,380 | 3.64 | 38,200 | 4.58 |
| 1937 | 3,310 | 3.52 | 7,920 | 3.90 |
| 1938 | 23,000 | 4.36 | 93,000 | 4.97 |
| 1939 | 1,260 | 3.10 | 3,230 | 3.51 |
| 1940 | 11,400 | 4.06 | 60,200 | 4.78 |
| 1941 | 12,200 | 4.09 | 30,300 | 4.48 |
| 1942 | 11,000 | 4.04 | 35,100 | 4.55 |
| 1943 | 6,970 | 3.84 | 54,300 | 4.73 |
| 1944 | 3,220 | 3.51 | 8,460 | 3.93 |
| 1945 | 3,230 | 3.51 | 28,600 | 4.46 |
| 1946 | 6,180 | 3.79 | 22,000 | 4.34 |
| 1947 | 4,070 | 3.61 | 17,800 | 4.25 |
| 1948 | 7,320 | 3.86 | 16,600 | 4.22 |
| 1949 | 3,870 | 3.59 | 6,140 | 3.79 |
| 1950 | 4,430 | 3.65 | 17,900 | 4.25 |
| 1951 | 3,870 | 3.59 | 50,200 | 4.70 |
| 1952 | 5,280 | 3.72 | 21,000 | 4.32 |
| 1953 | 7,710 | 3.89 | 40,000 | 4.60 |
| 1954 | 4,910 | 3.69 | 22,900 | 4.36 |
| 1955 | 2,480 | 3.39 | 5,900 | 3.77 |
| 1956 | 9,180 | 3.96 | 104,000 | 5.02 |
| 1957 | 6,140 | 3.79 | 32,700 | 4.51 |
| 1958 | 6,880 | 3.84 | 39,300 | 4.59 |

from Beard, 1962

By using 17 additional years of record at the base station, the short period of record is adjusted to

$$n_1' = \frac{30}{1-(47-30)/47\,(0.674)} = 39.7 \text{ YEARS}$$

Through the use of transposition of gaged data, the record at the short record gage has been effectively increased by approximately 10 years.

4.3.8 Sequence of Flood Frequency Calculations

The above sections have discussed several standard frequency distributions and a variety of adjustments to improve on the predictions and/or to account for unusual variations in the data. In most cases, not all the adjustments are necessary, and generally only one or two may be indicated. Whether the adjustments are even made may well depend on the size of the project and the purpose for which the data may be used.

For some of the adjustments, there is a preferred sequence of calculation, or in other words, there are some adjustments that must be made before others can be made. Bulletin 17B presents a flow chart outlining a path through the frequency calculations and adjustments. This outline forms the basis for many of the log-Pearson III computer programs such as J407 described above.

The SCS Handbook, 1972, also outlines the sequence for flood frequency analysis which is summarized as follows:

1. Obtain site information, the systematic station data, and historic information. This data should be examined for changes in watershed conditions, gage datum, flow regulation, etc. It is in this initial step that missing data should be estimated if indicated by the project.

2. Order the flood data, determine the plotting position, and plot the data on selected probability graph paper (usually log-probability). Examine the data trend to select the standard distribution/s that bestdescribe the population from which the sample is taken. Use a mixed population analysis if indicated by the data trend and the watershed information.

3. Compute the sample statistics and the frequency curve for the selected distribution. Plot the frequency curve with the station data to determine how well the flood data is distributed according to the selected distribution.

4. Check for high and low outliers. Adjust for historic data, retain or eliminate outliers and recompute the frequency curve.

5. Adjust data for missing low flows and zero flows and recompute the frequency curve.

6. Check the resulting frequency curve for reliability.

### 4.3.9 Other Methods for Estimating Flood Frequency Curves

The techniques of fitting an annual series of flood data by the standard frequency distributions described above are all samples of the application of the method of moments. Population moments are estimated from the sample moments with the mean, taken as the first moment about the origin, the variance as the second moment and the skew as the third moment.

There are three other recognized methods by which frequency curves can be determined. They include the method of maximum likelihood, regression equations and a graphical method. The method of maximum likelihood is a statistical technique based on the principle that the values of the statistical parameters of the sample are maximized so that the probability of obtaining an observed event is as high as possible. The method is somewhat more efficient for highly skewed distributions, if in fact efficient estimates of the statistical parameters exist. On the other hand, the method is very complicated to use and its practical use in highway design is not justified in view of the wide acceptance and use of the method of moments for fitting data with standard distributions. The method of maximum likelihood is described in detail by Kite, 1977, and appropriate tables are presented from which the standard distributions can be determined.

Least squares regression equations can also be fit to a set of annual flood data. The least squares method minimizes the sum of the squares of the difference between the observed and predicted values. Three conditions must be satisfied for efficiency of the least squares method. The deviations between the observed and predicted values are normally distributed, the variance of the deviations is independent along the fitted curve and the variance of the deviation is constant. These conditions are rarely met in highway design.

Graphical methods involve simply fitting a curve to the sample data by eye. Typically the data are transformed by plotting on probability or log-probability graph paper so that a straight line can be obtained. This procedure is the least efficient, but as noted in Sanders, 1980, some improvement is obtained by ensuring that the maximum positive and negative deviations from the selected line are equal and that the maximum deviations are made as small as possible. This is, however, an expedient method by which highway designers can obtain a frequency distribution estimate.

## 5.0 PEAK FLOW DETERMINATIONS FOR UNGAGED SITES

At many stream crossings of interest to the highway engineer, there may be insufficient stream gaging records, or often no records at all available for fitting a standard frequency distribution. In these cases, data from nearby watersheds with comparable hydrologic and physiographic features must be utilized.

Such procedures are often referred to as regional flood frequency methods and include:

1. Regional or other Regression Equations

2. Regional Analysis Methods

3. Peak Flow Formulas

### 5.1 Regional Regression Equations

Regional Regression Equations are the most commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Regression equations are used to relate either the peak flow or some other flood characteristic at a specified return period to the physiographic, hydrologic, and meteorologic characteristics of the watershed.

The typical multiple linear regression model utilized in regional flood studies is

$$Y_t = a \; X_1^{b_1} \; X_2^{b_2} \cdots X_n^{b_n} \tag{5-1}$$

Where $Y_t$ is the dependent variable, $X_1, X_2, \ldots, X_n$ are independent variables, a is the regression constant and $b_1, b_2, \ldots, b_n$ are regression coefficients. The dependent variable is normally taken to be the peak flow for a given return period or some other property of the particular flood frequency, and the independent variables are selected to characterize the watershed and its meteorologic conditions. The parameters a, $b_1$, $b_2, \ldots, b_n$ are determined in the regression analysis. Regression analysis is described in detail by Sanders, 1980, and Riggs, 1968.

The primary watershed characteristic is the drainage area and almost all regression formulas include drainage area above the point of interest as an independent variable. The choice of the other watershed characteristics is much more varied and can include measurements of channel slopes, lengths and geometry, shape factors, perimeter, basin fall, land use, among others. Meteorological characteristics often considered as independent variables include various rainfall parameters, snowmelt, evaporation, temperature, and wind. As many independent variables as desired can be used in a regression analysis although it would be unlikely that more than one measure of any particular characteristic would be included. The statistical significance of each independent variable can be determined and those that are statistically insignificant at a specified confidence level, e.g. the 95 percent confidence level, can be eliminated.

### 5.1.1 USGS Regression Equations

In a series of studies by the U.S. Geological Survey, the Federal Highway Administration and State Highway and other Departments, statewide regression equations have now been developed throughout the United States. These equations permit peak flows to be estimated for return periods varying from 2 to 100 years. Sauer et al., 1983, present the most current bibliography of state by state regional flood studies. References to these studies are summarized in Appendix D.

Typically, each state has been divided into regions of similar hydrologic, meteorologic and physiographic characteristics as determined by various statistical measures cited in Sec. 4.2.3. Using a combination of measured data and rainfall-runoff simulation models such as that of Dawdy et al., 1972, long-term records of peak annual flow were synthesized for each of several watersheds in a defined region. Each record was subjected to a log-Pearson Type III frequency analysis, adjusted as required for loss of variance due to modeling, and the peak flow for various frequencies determined.

Multiple linear regression was then used on the logarithmic transformed values of the variables to obtain regression equations of the form of Equation (5-1) for peak flows of selected frequencies. Only those independent variables that were statistically significant at predetermined confidence limits were retained in the final equations.

To illustrate the use of regional regression equations for estimating peak flows, consider the following example.

> It is desired to renovate a bridge at a highway crossing of the Seco Creek at D'Hanis, TX. The site is ungaged and the design return period is 25 years.
>
> The site lies in Region 5 as defined by Schroeder and Massey, 1977, and the applicable regression equations for this region are given as:

$$Q_2 = 4.82\ A^{0.799}\ S_o^{0.966} \qquad S_T(\%) = 62.1$$

$$Q_5 = 36.4\ A^{0.776}\ S_o^{0.706} \qquad S_T(\%) = 46.6$$

$$Q_{10} = 82.6\ A^{0.776}\ S_o^{0.622} \qquad S_T(\%) = 42.6$$

$$Q_{25} = 180\ A^{0.776}\ S_o^{0.554} \qquad S_T(\%) = 41.3$$

$$Q_{50} = 278 \quad A^{0.778} \, S_o^{\,0.522} \qquad\qquad S_T(\%) = 42.0$$

$$Q_{100} = 399 \quad A^{0.782} \, S_o^{\,0.497} \qquad\qquad S_T(\%) = 44.1$$

Where $Q_t$ is the peak annual flow for the specified return periods in CFS; A is the drainage area contributing surface runoff above the site in sq mi, $S_o$ is the average slope of the streambed between points 10 and 85 percent of the distance along the main stream channel from the site to the watershed divide in feet per mile, and $S_T$ (%) is the standard error in percent. The range of application of the above equations has been specified as:

1.08 < Drainage Area (sq mi) < 1947

9.15 < Slope (ft per mi) < 76.8

By planimetering the drainage area above the site from a topographic map, the area A is found to be 210.7 sq mi and the channel slope between the 10 and 85 percent points is 14.95 feet per mile. The 25-year peak flow is calculated to be

$$Q_{25} = 180 \, A^{0.776} \, S_o^{0.554} = 180 \, (210.7)^{0.776} \, (14.95)^{0.554}$$

$$Q_{25} = 51{,}190 \text{ CFS } (1450 \text{ CMS})$$

In most cases, regional regression equations are given with associated standard errors which are indicators of how accurately the regression equation predicts the observed data used in their development. The standard error of regression is a measure of the deviation of the observed data from the corresponding predicted values and is given by the basic equation

$$S_T = \left[ \frac{\sum_{i=1}^{n} (Q_i - \hat{Q}_i)^2}{n} \right]^{0.5} \tag{5-2}$$

where $Q_i$ is the observed value of the dependent variable (discharge) and $\hat{Q}_i$ is the corresponding value predicted by the regression equation. In a manner analogous to variance, the standard error can be expressed as a percentage by dividing Equation (5-2) by the mean value of the dependent variable, or

$$S_T\,(\%) = \frac{S_T}{\bar{Q}} \times 100 \tag{5-3}$$

The standard error of regression has a very similar meaning to that of the standard deviation, Equation (4-12), for a normal distribution in that approximately 68 percent of the observed data will be contained within ± one standard error of the regression line.

In order to better estimate the population standard error from a small sample, Equation (5-2) is writted as

$$S_T = \left[\frac{\sum_{i=1}^{n}(Q_i - \hat{Q}_i)^2}{n-m}\right]^{0.5} \tag{5-4}$$

where m is the number of variables (dependent and independent) in the regression equation or the number of regression coefficients (constants and exponents) determined in the analysis. For example, if a regression equation is determined between peak flow and drainage area, m = 2. In the above USGS regession equations for Region 5 in Texas, Q is given as a function of A and $S_o$, so m = 3.

Riggs, 1968, provides a comprehensive discussion of the Doolittle method for solving the simultaneous equations necessary to determine the regression coefficients and computing the standard error of estimate. To illustrate the standard error computation, consider the 25-year peak flow equation used in the above example for Seco Creek, Texas. This regression equation was given as

$$Q_{25} = 180\, A^{0.776}\; S_o^{0.554} \tag{5-5}$$

or in logarithmic form as

$$Q_{L,25} = 2.255 + 0.776\, A_L + 0.554\, S_{o,L} \tag{5-6}$$

where the subscripted "L" variables are the base 10 logarithms of the original variables.

The standard error is obtained by rewriting equation (5-4) with values of $\hat{Q}$ substituted from equation (5-6) as follows

$$S^2 = \frac{\Sigma(x_1^2) - b_2\,\Sigma(x_1 x_2) - b_3\,(x_1 x_3)}{n-m} \tag{5-7}$$

where $b_2$ = 0.776, $b_1$ = 0.554, m = 3 and

$$\Sigma(x_1^2) = \Sigma(Q_L)^2 - n(\bar{Q}_L)^2$$

$$\Sigma(x_1 x_2) = \left[\Sigma(Q_L)(A_L)\right] - n\,\bar{Q}_L\,\bar{A}_L$$

$$\Sigma(x_1 x_3) = \left[\Sigma(Q_L)(S_{QL})\right] - n\,\bar{Q}_L\,\bar{S}_{QL}$$

In the development of Equation (5-5), synthesized values of the 25-year peak flow for 27 stations in Region 5, Texas were used. These values together with the corresponding drainage areas and slopes are tabulated in Table 26. Also summarized in Table 26 are the values necessary to solve Equation (5-7).

Table 26. Region 5, Texas Data for Example Standard Error Computation

| STATION | Q (cfs) | A (sq mi) | $S_o$ (ft/mi) | $Q_L$ | $A_L$ | $S_L$ |
|---|---|---|---|---|---|---|
| 08160000 | 48440 | 114. | 28.3 | 4.68520 | 2.05690 | 1.45179 |
| 08167000 | 99660 | 838. | 13.3 | 4.99852 | 2.92324 | 1.12385 |
| 08167500 | 73600 | 1315. | 9.15 | 4.86688 | 3.11893 | 0.96142 |
| 08167600 | 14950 | 10.9 | 66.9 | 4.17464 | 1.03743 | 1.82543 |
| 08171000 | 58410 | 355. | 17.2 | 4.76649 | 2.55023 | 1.23553 |
| 08171800 | 98160 | 412. | 13.6 | 4.99193 | 2.61490 | 1.13354 |
| 08178600 | 12820 | 9.54 | 41.73 | 4.10789 | 0.97955 | 1.62045 |
| 08179000 | 67300 | 474. | 16.2 | 4.82802 | 2.67578 | 1.20952 |
| 08179100 | 28630 | 56.3 | 36.4 | 4.45682 | 1.75051 | 1.56110 |
| 08181200 | 1170 | 1.08 | 76.8 | 3.06819 | 0.03342 | 1.88536 |
| 08181400 | 4610 | 15. | 49.5 | 3.66370 | 1.17609 | 1.69461 |
| 08182400 | 6010 | 7.01 | 35.4 | 3.77887 | 0.84572 | 1.54900 |
| 08183900 | 38240 | 68.4 | 24.29 | 4.58252 | 1.83506 | 1.38543 |
| 08185000 | 53810 | 274. | 12.9 | 4.73086 | 2.43775 | 1.11059 |
| 08190000 | 196100 | 764. | 14.8 | 5.29248 | 2.88309 | 1.17026 |
| 08190500 | 192400 | 700. | 13.8 | 5.28421 | 2.84510 | 1.13988 |
| 08192000 | 237600 | 1947. | 10.4 | 5.37585 | 3.28937 | 1.01703 |
| 08195000 | 91520 | 405. | 20. | 4.96152 | 2.60746 | 1.30103 |
| 08196000 | 71690 | 117. | 25. | 4.85546 | 2.06819 | 1.39794 |
| 08198000 | 47250 | 206. | 22.5 | 4.67440 | 2.31387 | 1.35218 |
| 08198500 | 66000 | 247. | 18.2 | 4.81954 | 2.39270 | 1.26007 |
| 08198900 | 6510 | 10.6 | 16.87 | 3.81358 | 1.02531 | 1.22712 |
| 08200000 | 53890 | 86.2 | 32.7 | 4.73151 | 1.93551 | 1.51455 |
| 08200500 | 62250 | 132. | 22.6 | 4.79414 | 2.12057 | 1.35411 |
| 08201500 | 35480 | 43.1 | 34.7 | 4.54998 | 1.63448 | 1.54033 |
| 08202500 | 36740 | 87.4 | 27.9 | 4.56514 | 1.94151 | 1.44560 |
| 08202700 | 56960 | 168. | 20.32 | 4.75557 | 2.22531 | 1.30792 |
| | | | SUM | 124.17390 | 55.31795 | 36.77563 |
| | | | MEAN | 4.59903 | 2.04881 | 1.36206 |

For this sample data

$$\Sigma\,(x_1^2) = (578.41) - (27)(4.5990)^2$$

$$\Sigma\,(x_1\,x_2) = (264.68) - (27)(4.5990)(2.0488)$$

$$\Sigma\,(x_1\,x_3) = (166.66) - (27)(4.5990)(1.3621)$$

and from Equation (5-7), the standard error of estimate is

$$S_T^2 = \frac{7.3296 - 7.9711 + 1.3719}{27 - 3} = 0.03043$$

or

$$S_T = 0.17445 \quad (\text{in log units})$$

The standard error, in percent, is determined from the antilogs of $(1 + S_T)$ and $(1 - S_T)$ which are then taken as ratios to 10 to obtain the percentage deviation or

$$\frac{\text{ANTILOG}\,(1 + S_T) - 10}{10} \times 100 = +\ \text{PERCENT DEVIATION} \tag{5-8}$$

and

$$\frac{10 - \text{ANTILOG}\,(1 - S_T)}{10} \times 100 = -\ \text{PERCENT DEVIATION} \tag{5-9}$$

In the above example, antilog$(1 + S_T)$ = antilog(1 + 0.17445) = 14.95 and the antilog$(1 - S_T)$ = antilog(1 - 0.17445) = 6.69. The percentage deviations are then computed as

$$\frac{(14.95 - 10)}{10} \times 100 = 49.5\ \%$$

and

$$\frac{(10 - 6.69)}{10} \times 100 = 33.1\ \%$$

The average standard error in percent is taken as the average percentage deviation which for the above example is 41.3 percent, the reported value for this particular regression equation. When computed for a log transformed dependent variable, (log Q), the standard error represents a constant percentage of the regressed curve value as contrasted to a constant mangnitude when computed with untransformed values. If the standard error for the above example is computed using linear values with Equations (5-4) and (5-3), its value is 44.7 percent, reflecting this difference in interpretation.

Because of the extensive use now being made of USGS regression equations, it is of interest to compare peak discharges estimated from these equations with results obtained from a formal flood frequency analysis as described in Sec. 4.0. A direct comparison cannot be made with the previously used Medina River data because of some storage and regulation upstream of the gage. Since regression equations apply only to totally unregulated flow , Station 08179000, Medina River near Pipe Creek, Texas has been selected for comparison. This gage has 43 years of record, drains an area of 474 sq mi, is totally unregulated and has station and generalized skews of -0.005 and -0.234 respectively. Using the USGS program J407, the data was analyzed with a log-Pearson III distribution, and the 10-, 25-, 50- and 100-year peak discharges estimated using the Bulletin 17B weighted skew option ($G_L$=-0.227). These values together with peak flows determined from a frequency curve through the systematic record, are summarized in Table 27.

The Pipe Creek gage is located in Region 5 in Texas and the regression equations given for the Seco Creek example above are applicable. The watershed has an average slope of 16.2 ft per mi between 10 and 85 percent points along the main stream channel. The corresponding peak flows calculated from the appropriate regression equations are also summarized in Table 27.

Table 27. Comparison of Peak Flows from Log-Pearson Type III Distribution and USGS Regional Regression Equation

| Return Period | Peak Discharge - CFS | | |
|---|---|---|---|
| | Log Pearson III Frequency | Systematic Record | USGS Regression Equations |
| 10-year | 42,628 | 50,258 | 62,226 |
| 25-year | 68,814 | 88,969 | 100,414 |
| 50-year | 92,861 | 128,637 | 143,614 |
| 100-year | 120,816 | 179,194 | 196,932 |

The peak discharges estimated from the regression equations are all substantially higher than the comparable values determined from the log-Pearson III analysis, although all are within the Bulletin 17B, upper 95-percent confidence limits. Further review of the data at this station indicates that a frequency curve constructed using the systematic record plots above the log Pearson III distribution curves at least over the range of frequencies considered in the above comparison. This is partially a result of a peak flow in 1978 in excess of 281,000 CFS (7958 CMS) which according to the log-Pearson III analysis is an event approaching the 500-year peak flow.

It has been suggested by some experienced hydrologists that regression equations may give better estimates of peak flows of various frequencies than formal statistical frequency analyses. They reason that regression equations more nearly reflect the potential or capacity of the watershed to experience a peak flow of given magnitude whereas a frequency analysis is biased by what has been recorded at a gage. There is some justification for this argument as there are many examples throughout the country of adjacent watersheds of comparable size and physiographic and hydrologic characteristics wherein only one has recorded major floods. This is obviously a function of where the storm occurs, but frequency analyses of gaged data from the different watersheds may give very different peak flows for the same frequencies. On the other hand, regression equations will give comparable flood magnitudes at the same frequencies for each watershed, all other factors being approximately equal, regardless of in which watershed the storm occurs.

This is not to suggest that regional regression equations should take precedence over frequency analysis especially when sufficient data are available. Regression equations, however, do serve as a basis for comparison of statistically determined peak flows of specified frequencies and provide for further evaluation of the results of a frequency analysis. They may be used to add credence to historical flood data or may indicate that historical records should be sought out and incorporated into the analysis. Regression equations can provide insight into the treatment of outliers beyond the purely statistical methods discussed in Sec. 4.3.7.1. As demonstrated by the above discussion, comparison of the peak flows obtained by different methods may well indicate the need to review data from other comparable watersheds within a region and the desirability of transposing or extending a given record using data from other gages.

There are several points that should be kept in mind when using regional regression equations. For the most part, the state regional equations are developed for unregulated, natural, nonurbanized watersheds. They separate out mixed populations, i.e. rain produced floods from snowmelt floods or hurricane associated storms. The equations are regionalized so that it is incumbent on the user to carefully define the hydrologic region and to define the dependent and independent variables in the exact manner prescribed for each set of regional equations. The designer is also cautioned to apply these equations within the range of independent variables utilized in the development of the equations.

Although not a serious problem, the designer should be alert to any discrepancies in results from regression equations when applied at regional boundaries and especially near state boundaries. Within-state regional boundaries generally define hydrologic regions with similar characteristics, and regression equations may not give comparable results near regional boundaries. Hydrologic regions also may cross state boundaries, and regression equations for adjacent regions in different states can give substantially different peak flows for the same frequency. When working near within-state regional and state boundaries, regression equations for adjacent regions should be checked and any serious discrepancies justified.

It should be noted that in some cases, there are regions within a State for which regression equations are not available. These areas result from either insufficient data, lack of definition of the flood frequency characteristic of mixed storm events, and in cases where there are numerous natural lakes, the inability to properly define the contributing drainage area. Also, separate urban studies have been conducted in some metropolitan areas which present more applicable regression equations than those discussed above. These urban studies are listed in Section 8.0 of this manual.

### 5.1.2 FHWA Regression Equations

In 1977, the Federal Highway Administration published a two-volume report by Fletcher, et al. which presents nationwide regression equations for predicting runoff from small rural watersheds (<50 sq mi). This method is not the equivalent of the regression equations described above, and consequently has not been used as extensively as the USGS regional peak flow equations. The procedure is similar in concept to that of Potter, 1961, and uses frequency analysis of data in over 1000 small watersheds throughout the United States and Puerto Rico to relate peak flows to various hydrographic and physiographic characteristics. Three-, five-, and seven-parameter regression equations were developed for the 10-year peak runoff for each of 24 hydrophysiographic regions. Since the standard errors of estimate were found to be approximately the same for each regression equation option, the following discussion is limited to the three-parameter equations only.

If a drainage structure is to be designed to carry the probable maximum flood peak, $Q_{p(max)}$ in CFS, Fletcher, et al. give the equation

$$Q_{P(MAX.)} = 10^{\left[3.92 + 0.812\,(\mathrm{Log}\,A) - 0.0325\,(\mathrm{Log}\,A)^2\right]} \qquad (5\text{-}10)$$

where log A is the logarithm to the base 10 of the drainage area in square miles. If it is feasible to construct a very large drainage structure to handle this probable maximum flow, the hydrologic analysis is essentially complete. Similarly, if a minimum size drainage structure is specified, and its carrying capacity is greater than $Q_{p(max)}$, no further analysis is required.

A more common problem in highway drainage is that the structure must be designed to handle a flow of specified frequency. This can be accomplished with the three-parameter FHWA regression equations. The basic form of these equations is

$$\hat{q}_{10} = a\,(A)^{b_1}\,(R)^{b_2}\,(DH)^{b_3} \qquad (5\text{-}11)$$

Where $\hat{q}_{10}$ is the 10-year peak runoff in CFS, A is the drainage area in sq mi, R is the isoerodent factor defined as the product of the mean annual rainfall kinetic energy and the maximum respective 30-minute annual maximum rainfall intensity, DH is the difference in elevation measured along the main drainage structure site in feet, and a, $b_1$, $b_2$, and $b_3$ are obtained from the regression analysis. Values of the drainage area and elevation difference are readily determined from topographic maps and R is taken from individual state isoerodent maps given by Fletcher et al.

Two options are available to use the three-parameter regression equations. The first involves the application of an equation of the same form as Equation (5-11) for a specific hydrophysiographic zone. Twenty-four zones are defined covering the United States and Puerto Rico and each has its own regression equation for $\hat{q}_{10}$. The second option involves the use of an all zone equation developed from all the data. The all zone three-parameter equation for the 10-year peak dischage, $\hat{q}_{10(3AZ)}$, is

$$\hat{q}_{10(3AZ)} = 1.28015\,(A)^{0.56172}\,(R)^{0.94356}\,(DH)^{0.16887} \qquad (5\text{-}12)$$

For each of the 24 hydrophysiographic zones, there is a correction equation presented to adjust Equation (5-12) for zonal bias. These correction equations are all of the form

$$\hat{q}_{10} = a_1\,\hat{q}_{10(3AZ)}^{b_1} \qquad (5\text{-}13)$$

where $a_1$ and $b_1$ are again appropriate regression coefficients. If the area surface water storage is more than about 4 percent of the total drainage area, it is recommended that the value of $\hat{q}_{10}$ computed from an individual zone equation or the corrected all-zone equation be further adjusted with a storage correction multiplier given with the equations.

Fletcher et al. then present the following equations from which a frequency curve can be drawn on any appropriate probabilty paper

$$Q_{2.33} = 0.46921\,\hat{q}_{10}^{1.00243} \qquad (5\text{-}14)$$

$$Q_{50} = 1.45962 \ \hat{q}_{10}^{1.02342} \tag{5-15}$$

$$Q_{100} = 1.64380 \ \hat{q}_{10}^{1.02918} \tag{5-16}$$

where $Q_{2.33}$ is the mean annual peak flow taken at a return period of 2.33 years and $Q_{50}$ and $Q_{100}$ are the 50- and 100-year peak flows respectively. From this curve, the flow for any other selected design frequency can be determined.

The concept of risk can also be incorporated into the FHWA regression equations. Recall that risk is the probability that one or more floods will exceed the design discharge within the life of the project. Methods presented by Fletcher et al. permit the return period of the design flood to be adjusted according to the risk the designer can accept. The concept of the probable maximum peak flow is also useful because it represents the upper limit of flow that might be expected. It can, therefore, have application to situations where the consequences of failure are very large or unacceptable.

## 5.2 Regional Analysis Methods

Other methods exist for determining peak flows for various exceedance frequencies using regional methods where no data are available. These include

1. USGS Index-Flood Method
2. Regionalization of Parameters

### 5.2.1 USGS Index-Flood Method

The Index-Flood method of regional analysis described by Dalrymple, 1960, was used extensively in the 1960's and early 1970's. This method utilizes statistical analyses of data at meteorologically and hydrologically similar gages to develop a flood frequency curve at an ungaged site. There are two parts to the Index-Flood method. The first consists of developing the basic dimensionless ratio of a specified frequency flow to the index flow (usually mean annual flood) and the second involves developing the relation between the drainage basin characteristics (usually drainage area) and the mean annual flood.

The procedure to develop a regional flood frequency curve by the Index-Flood method is described by the following 11 steps.

1. Tabulate annual peak floods for all gages within the hydrologically similar region.
2. Select the base period of record. This is usually taken as the longest period of record.

3. Estimate floods for missing years by correlation with other data.

4. Assign an order to all floods (actual and estimated) at each station, compute the plotting position and plot frequency curves using the best standard distribution fit for each gage. These frequency curves should have about equal slopes.

5. Determine the mean annual floods for each gage as the discharge with a return period of 2.33 years. This is a graphical mean which is more stable than the arithmetic mean and its value is not affected as much by the inclusion or exclusion of major floods. It also gives a greater weight to the median floods than to the extreme floods where sampling errors may be larger.

6. Test the data for homogeneity. This is accomplished in the following manner.

   a. For each gage, compute the ratio of the flood with a 10-year return period, $Q_{10}$, to the station mean, $Q_{2.33}$. (Both of these values are obtained from the frequency analysis).

   b. Compute the arithmetic average of the ratio $Q_{10}/Q_{2.33}$ for all the gages considered.

   c. For each gage, compute $Q_{2.33}\,(Q_{10}/Q_{2.33})_{avg}$ and the corresponding return period.

   d. Plot the values of return period obtained in step c. against the effective length of record, $L_E$, for each gage

   (5-17)

   where L is the actual length of record at a gage and $L_B$ is the length of the base record.

   e. Test for homogeneity by also plotting on this graph, envelope curves determined from Table 28 below, taken from Dalrymple, 1960. This Table gives the upper and lower limits, $T_u$ and $T_L$, as a function of the effective length of record.

   (Table 28 applies only to homogeneity tests of the 10-year floods). This homogeneity test is illustrated in Figure 35 on Gumbel probability paper (USGS Form 9-179a.) .

Table 28. Upper and Lower Limit Coordinates of Envelope Curve for Homogeneity Test

| Effective Length of Record, $L_E$ (YRS) | Return Period Limits, Tr (YRS) | |
|---|---|---|
| | Upper Limit | Lower Limit |
| 5 | 160 | 1.2 |
| 10 | 70 | 1.85 |
| 20 | 40 | 2.8 |
| 50 | 24 | 4.4 |
| 100 | 18 | 5.6 |

from Dalrymple, 1960

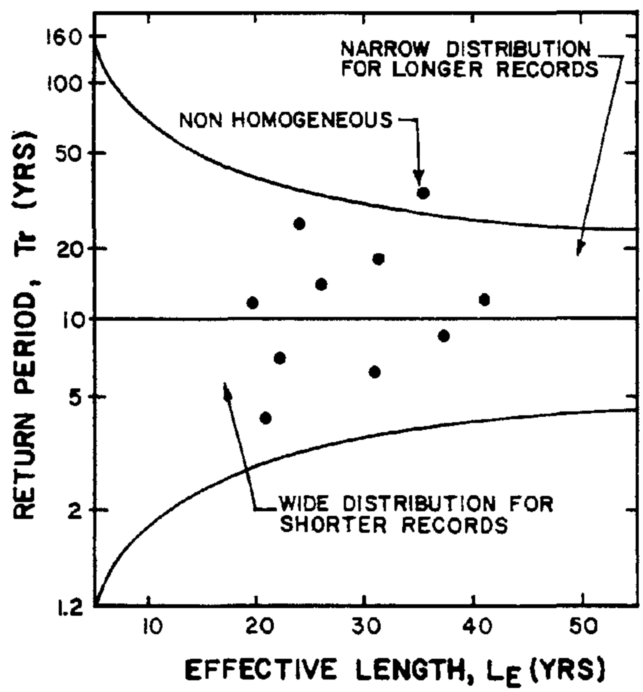



Figure 35. Hydrologic Homogeneity Test

Return periods which fail this homogeneity test should be eliminated from the regional analysis.

7. Using actual flood data, compute the ratio of each flood to the station mean, $Q_{2.33}$, for each record.

8. Compute the median flood ratios of the stations retained in the regional analysis for each rank or order m, and compute the corresponding return period by the Weibull Formula, $Tr = (n+1)/m$. (It is suggested that the median ratio be determined after eliminating the highest and lowest $Q/Q_{2.33}$ values for each ordered series of data).

9. Plot the Median Flood Ratio against the return period on probability paper.

10. Plot the logarithm of the mean annual flood for each gage, $Q_{2.33}$ against the logarithm of the corresponding drainage area. This curve should be nearly a straight line.

11. Determine the flood frequency curve for any stream site in the watershed as follows:

    a. Determine the drainage area above the site.

    b. From step 10, determine the value of $Q_{2.33}$.

    c. For selected return periods, multiply the median flood ratio in step 9 by the value of $Q_{2.33}$ from step 11b.

    d. Plot the regional frequency curve.

Example problems illustrating the Index Flood method are contained in Dalrymple, 1960, Sanders, 1980, and numerous hydrology textbooks.

As pointed out by Benson, 1962, the Index-Flood method has some limitations which can affect its reliability. The most significant is that there may be large differences in the index or mean annual floods throughout a region. This can lead to considerable variations in the various flood ratios even for watersheds of comparable size. Another shortcoming of the method is that homogeneity is established at the 10-year level, whereas at the higher levels the test may not be sustained. Still another deficiency pointed out by Benson is that all sizes of drainage areas (except the very largest) are included in the Index-Flood regional analysis. As discussed in Section 2 of this manual, the larger the drainage area, the flatter the frequency curve will be. This effect is most noticeable at the higher return periods.

During the period 1964-1968, the U.S. Geological Survey utilized the Index-Flood Method to provide a means for estimating the magnitude and frequency of floods at gaged and ungaged sites throughout the United States. The results of these studies are published in 19 Water Supply Papers under the general title "Magnitude and Frequency of Floods in the United States" and each covers a specific hydrologic region. Table 29 is a summary of these 19 reports and gives the Water Supply Paper Number, the hydrologic region covered and the date of the publications.

Table 29. Summary of USGS Water Supply Papers Utilizing Index-Flood Method for Estimates of Magnitude and Frequency of Floods

| WSP No. | Hydrologic Region | Date |
|---|---|---|
| 1671 | Part 1A. North Atlantic Slope Basins, Maine to Connecticut, A.R. Green | 1964 |
| 1672 | Part 1B. North Atlantic Slope Basins, New York to York River, R.H. Tici | 1968 |
| 1673 | Part 2A. South Atlantic Slope Basins, James River to Savannah River, P.R. Speer & C.R. Gamble | 1964 |
| 1674 | Part 2B. South Atlantic Slope Basins and Eastern Gulf of Mexico Basins, Ogeechee River to Pearl River, H.H. Barnes, Jr. & H.G. Golden | 1966 |
| 1675 | Part 3A. Ohio River Basin except Cumberland and Tennessee River Basins, P.R. Speer & C.R. Gamble | 1965 |
| 1676 | Part 3B. Cumberland and Tennessee River Basins, P.R. Speer & C.R. Gamble | 1964 |
| 1677 | Part 4. St. Lawrence River Basin, S.W. Wiitala | 1965 |
| 1678 | Part 5. Hudson Bay - Upper Mississippi River Basin, J.L. Patterson & C.R. Gamble | 1968 |
| 1679 | Part 6A. Missouri River Basin above Sioux City, Iowa, J.L. Patterson | 1966 |
| 1680 | Part 6B. Missouri River Basin below Sioux City, Iowa, H.F. Mattahai | 1968 |
| 1681 | Part 7. Lower Mississippi River Basin, J.L. Patterson | 1964 |
| 1682 | Part 8. Western Gulf of Mexico Basins, J.L. Patterson | 1965 |
| 1683 | Part 9. Colorado River Basin, J.L. Patterson & W.P. Somers | 1966 |
| 1684 | Part 10. The Great Basin, E.B. Butler, J.K. Reid & V.K. Berwick | 1966 |

Table 29. Summary of USGS Water Supply Papers Utilizing Index-Flood Method for Estimates of Magnitude and Frequency of Floods (continued)

| WSP No. | Hydrologic Region | Date |
|---|---|---|
| 1685 | Part 11. Pacific Slope Basins of California, Vol. 1 Coastal Basin South of Klamath River Basin and Central Valley Drainage from the West, L.E. Young & R.W. Cruff | 1967 |
| 1686 | Part 11. Pacific Slope Basins of California, Vol. 2 Klamath and Smith River Basins and Central Valley Drainage from the East, L.E. Young & R.W. Cruff | 1967 |
| 1687 | Part 12. Pacific Slope Basins in Washington and Upper Columbia River Basin, G.L. Bodhaine & D.M. Thomas | 1964 |
| 1688 | Part 13. Snake River Basin, C.A. Thomas, H.C. Broom & J.E. Cummans | 1963 |
| 1689 | Part 14. Pacific Slope Basins in Oregon and Lower Columbia River Basins, Harry Hulsing & N.A. Kallio | 1964 |

With the development of regional regression equations for peak-flow in most states, there is only limited application of the Index-Flood method today. It is used primarily as a check on other solution techniques and for those situations where other techniques are inapplicable or not available.

### 5.2.2 Regionalization of Parameters

Beard, 1962, describes a regional flood frequency analysis for ungaged sites where the mean and standard deviation of the log annual series are related to the watershed characteristics by regression analysis. Lines of equal regression constants are plotted on a map from which values can be interpolated for the site of interest. The estimated regression constants can then be used to obtain the mean and standard deviation for the point of interest and various frequency flows determined from the standard frequency distribution. The method can be extended by regressing also on the generalized skew coefficient if a log-Pearson III distribution is desired. The detailed procedures for regionalizing statistical parameters are given by Beard and by Sanders, 1980.

## 5.3 Rational Formula

One of the most commonly used equations for the calculation of peak flow from small areas is the Rational Formula. In its most common form, the Rational Formula is given as

$$Q = CiA \tag{5-18}$$

where Q is the peak flow in CFS, i is the rainfall intensity in in/hr, A is the drainage area in acres, and C is a dimensionless runoff coefficient assumed to be a function of the cover of the watershed. While Equation (5-18) is a formula with mixed units, the conversion of the volume rate, inch-acres/hr to CFS is 1.008 so the error in units is 0.8 percent which is negligible compared to the other assumptions.

The assumptions in the Rational Formula are as follows:

1. Drainage area should be smaller than 300 acres.

2. Peak flow occurs when all of the watershed is contributing.

3. The rainfall intensity is uniform over a duration of time equal to or greater than the time of concentration, $T_c$. The time of concentration is the time required for water to travel from the most remote point of the basin to the outlet or point of interest.

4. The frequency of the peak flow is equal to the frequency of the rainfall intensity. In other words, the 10-year rainfall intensity, i, is assumed to produce the 10-year flood.

The runoff coefficient, C, is taken to be a function of ground cover only and is considered independent of the intensity of the rainfall. Actually, C is a volumetric coefficient which relates the peak discharge to the "theoretical peak" or 100 percent runoff. Hence C is also a function of infiltration and other hydrologic abstractions. Some typical values of C for the rational formula are given in Table 30. Should the basin contain varying amounts of different cover, a weighted runoff coefficient for the entire basin can be determined as

$$\text{Weighted } C = \frac{\Sigma C_i A_i}{A} \tag{5-19}$$

The construction of a rainfall intensity-duration-frequency curve requires a frequency analysis of rainfall amounts of various durations. The U.S. Weather Bureau, 1961, published a rainfall atlas for the United States in which isohyets of inches of rainfall are plotted throughout the United States for various frequencies and durations. From these data, it is possible to develop an intensity curve such as shown in Figure 36. Today, most agencies and city and county public works departments have updated the USWB Atlas data and have available intensity-duration-frequency curves for their respective jurisdictions.

Table 30. Runoff Coefficients for Rational Formula

| Type of Drainage Area | Runoff Coefficient |
|---|---|
| Business: | |
| Downtown areas | 0.70-0.95 |
| Neighborhood areas | 0.50-0.70 |
| Residential: | |
| Single-family areas | 0.30-0.50 |
| Multi-units, detached | 0.40-0.60 |
| Multi-units, attached | 0.60-0.75 |
| Suburban | 0.25-0.40 |
| Apartment dwelling areas | 0.50-0.70 |
| Industrial: | |
| Light areas | 0.50-0.80 |
| Heavy areas | 0.60-0.90 |
| Parks, cemeteries | 0.10-0.25 |
| Playgrounds | 0.20-0.40 |
| Railroad yard areas | 0.20-0.40 |
| Unimproved areas | 0.10-0.30 |
| Lawns: | |
| Sandy soil, flat, 2% | 0.50-0.10 |
| Sandy soil, average, 2-7% | 0.10-0.15 |
| Sandy soil, steep, 7% | 0.15-0.20 |
| Heavy soil, flat, 2% | 0.13-0.17 |
| Heavy soil, average 2-7% | 0.18-0.22 |
| Heavy soil, steep, 7% | 0.25-0.35 |
| Streets: | |
| Asphaltic | 0.70-0.95 |
| Concrete | 0.80-0.95 |
| Brick | 0.70-0.85 |
| Drives and walks | 0.75-0.85 |
| Roofs | 0.75-0.95 |

from ASCE, 1960

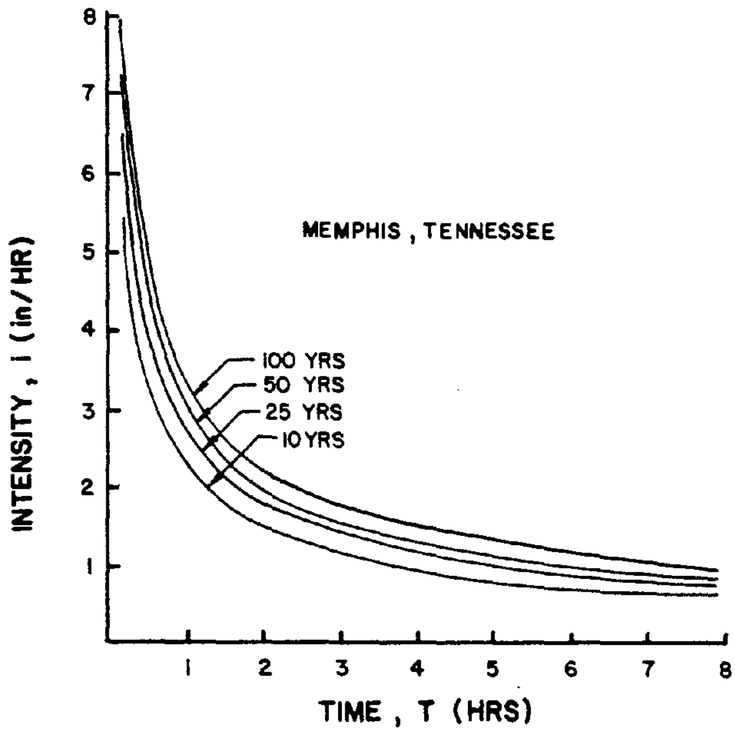



Figure 36. Rainfall Intensity-Duration-Frequency Curves, Memphis, Tennessee

The time of concentration, $T_c$, must be estimated from the basin characteristics and the description of the water course--concentrated or unconcentrated. For concentrated flow, the average flow velocity can be estimated from open channel and pipe flow equations whereas for an unconcentrated flow, average velocities can be calculated by overland flow methods. Figure 37 taken from the SCS Handbook, 1972, gives some approximate average velocities from which the time of concentration can also be estimated.

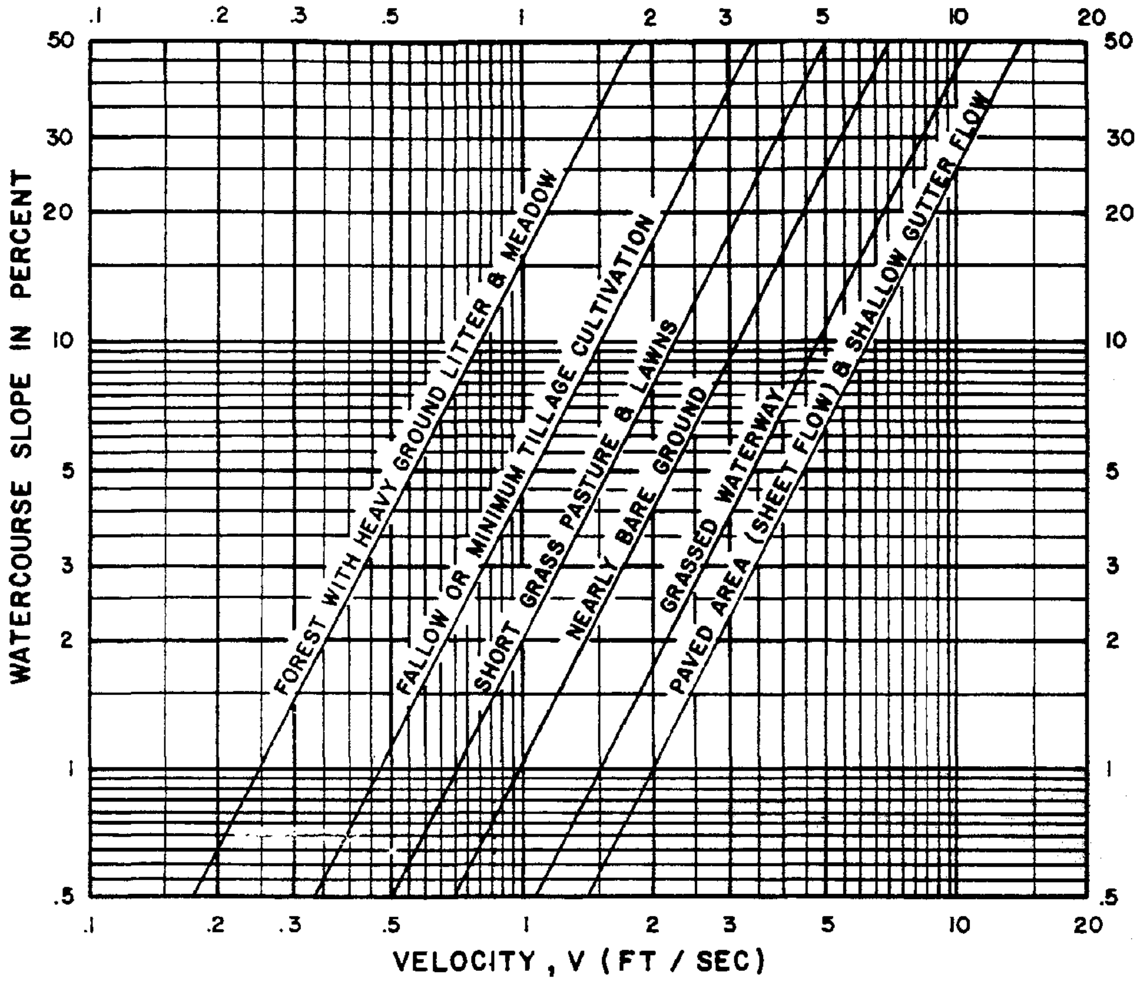
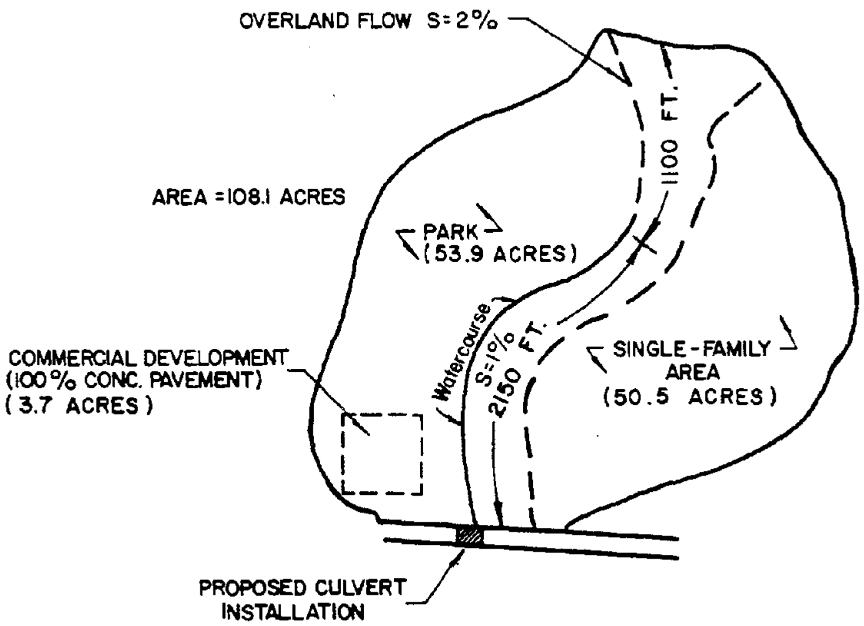



Figure 37. Velocities for Upland Method of Estimating Time of Concentration

As an illustration of the use of the Rational Formula consider the following example.

> A flooding problem exists along a farm road near Memphis, Tennessee. A low water crossing is to be replaced by a culvert installation to improve the road safety during rainstorms. The drainage area of the intermittent creek is as sketched below and has an area of 108.1 acres. The design storm is to be 25 years as determined by local authorities. Determine the maximum flow the culverts must pass for the indicated design storm.

DRAINAGE AREA




SCALE: 1"=1000'

1. Weighted "C" value - From the above sketch of the watershed and Table 30; a summary of "C" values by areas is prepared as shown.

| Description | C Value from Table 30 | Area (acres) | $C_i A_i$ |
|---|---|---|---|
| Park | .2 | 53.9 | 10.78 |
| Commercial Development | .95 | 3.7 | 3.52 |
| Single-family | .40 | 50.5 | 20.20 |
| | TOTALS | 108.1 | 34.50 |

$$\text{Weighted } C = \frac{\Sigma C_i A_i}{A} = \frac{34.50}{108.1} = \underline{\underline{0.32}}$$

2. Intensity - i The 25-year intensity is taken from the frequency curve in Figure 36. To obtain intensity the time of concentration, $T_c$, must first be estimated. In this example the hydraulic method for $T_c$ is used.

   A) Overland flow (1100 ft)--"Short Grass Pasture & Lawns" at 2 percent (Figure 37) : V = 1 ft/sec

   B) Channelized flow (2150 ft)--"Grassed Waterway" at 1 percent (Figure 37) : V = 1.5 ft/sec

   C) Time of Concentration is estimated as

$$T_c = \Sigma\left(\frac{D}{V}\right) = \frac{1100\,\text{ft}}{1\,\text{ft/sec}} + \frac{2150\,\text{ft}}{1.5\,\text{ft/sec}}$$

$$= 2533\ \text{sec} \div 3600\ \frac{\text{sec}}{\text{hr}} = 0.70\,\text{hrs}$$

   D) Intensity is obtained from Figure 36 using a duration equal to the time of concentration.

$$i = 3.3\ \text{in/hr}$$

3) Area - A Total area of drainage basin, A = 108.1 acres

$$\text{Peak Flow } Q_{25} = Ci_{25}A = (0.32)(3.3)(108.1)$$

$$Q_{25} = 114.2\ \text{CFS}\ (3.2\ \text{CMS})$$

## 5.4 Other Peak Flow Methods

There are many other methods for estimating peak flow for gaged and ungaged watersheds. These include graphical methods, formulas, tables, and combinations thereof. In most cases, these methods include empirically determined coefficients and exponents. They are highly regionalized, often applying only to a single watershed and to a limited range of flood peaks, and consequently have limited application. Therefore, the above discussions have been limited to the more generalized procedures which have been used throughout the United States and which have established and proven reliability.

There are, however, other accepted methods for peak flow determinations. They include design hydrographs which give a complete time history of the passage of a flood at a particular site including the peak flow. Hydrographs and their development for gaged and ungaged watershed are discussed in Section 6.0 of this manual. The Soil Conservation Service, 1972 and 1975, also presents curves from which peak flow can be graphically estimated for particular types of rainfall distributions. The methods also involve detailed calculations of the characteristics of the watershed. Both of these graphical methods are also discussed in Sections 6.0 and 8.0 of this manual.

## 5.5 Nationwide Test for Estimating Peak Flow Frequency at Ungaged Watersheds

In 1981, the Water Resources Council reported on the work of an interagency work group of the Hydrology Committee to develop national guidelines for defining peak flow frequencies at ungaged watersheds. The guidelines were to be selected from procedures currently in use based on the criteria of accuracy within acceptable standards, reproducibility of results by different people using the same procedures, and practicality or cost effectiveness.

Eight categories for estimating peak flow frequency were identified in the classification scheme. They included the following:

1. Statistical Estimation of Peak Flows for a Given Exceedance Probability. Regression equations for peak flow in terms of watershed and climatic conditions and frequency values from station data.

2. Statistical Estimation of Moments. Moments of a probability distribution of a series of peak flows (mean, standard deviation and skew) are related to watershed and climatic conditions through graphical and statistical methods.

3. Index Flood. Peak discharge estimates are estimated for different exceedance probabilities through appropriate index ratios.

4. Transfer Methods. Peak flows are extrapolated from peak flow values upstream and downstream of the point of interest or interpolated from other sites where frequency curves have been developed.

5. Empirical Equations. Peak flows are estimated from equations, such as the rational formula, or developed by methods other than regression analysis or hydrograph techniques.

6. Single Storm Event. Hydrographs are developed from storms of specified frequency and used to compute peak discharge assuming the peak discharge frequency is the same as the rainfall frequency.

7. Multiple Discrete Events. Watershed models are used to compute one or more peak floods per year using the largest rainfall events and a frequency curve is developed from the computed maximum floods.

8. Continuous Record. Continuous hydrographs are generated from watershed models using measured or synthetically developed continuous rainfall records and a frequency curve is obtained from simulated annual peak flows.

The major conclusions from this WRC study include the following. First, there is very limited published information comparing the performance of different procedures for estimating peak flows. The limited information reviewed found that at a given site, large differences in flood estimates by various procedures can be expected.

Secondly, there was no consensus on procedures among 7 federal agencies, state highway departments and the private sector. Table 31 taken from the WRC report, does, however, provide some insight into the relative use of the different procedures. This table summarizes the percentage of projects in which the various procedures have been used.

Table 31. Frequency of Use of Procedure Categories (in percent)

| Procedure Categories | Federal* Agencies | State Highway Departments | Private Sector |
|---|---|---|---|
| Statistical Estimation of $Q_p$ | 48 | 38 | 34 |
| Statistical Estimation by Moments | 1 | 0 | 4 |
| Index Flood Method | 1 | 4 | 3 |
| Transfer Method | 1 | 19 | 7 |
| Empirical Equations | 24 | 38 | 17 |
| Single Storm | 24 | 1 | 34 |
| Multiple Discrete Events | 1 | 0 | 0 |
| Continuous Record | 0 | 0 | 1 |

*Based on small samples, modest to important projects
from WRC, 1981

This table shows that extensive use is made of the state regression equations and other empirical formulas such as the Rational Formula by federal agencies, state highway departments and the private sector. The state highway departments make minimal use of hydrograph methods for single storms compared to federal and private use, opting perhaps for the transposition of data from nearby gages and watersheds. As pointed out earlier, the application of Index Flood methods are limited, and practically no use is made of watershed models for discrete and continuous hydrograph simulation. Since the study was conducted primarily for ungaged watersheds, the use of statistical estimation by moments is expected to be minimal.

Because of the many different procedures used in practice and the different opinions about their use, a nationwide test of procedure performance was prepared based on accuracy, reproducibility and practicality. A pilot test was conducted for 70 sites in the Mid- and Northwest. About 200 persons used up to 10 different procedures which resulted in about 1800 procedure applications.

The results confirmed that differences in procedure performance could be detected in terms of the performance criteria and that national guidelines could be developed. Writing in the Transportation Research Record, Newton and Herrin, 1982, concluded that while the test covered only a limited part of the country, "the USGS State Equations and Index Flood methods were found to be the most accurate and reproducible procedures evaluated". They attributed this superior performance to the definition of the parameters and the formulation of the prediction equations. Best performance was found when the parameters in the equations were uniquely defined and could be measured or determined consistently; the equations were formulated so that the frequency estimates were insensitive to variations in the parameters; and the equations were well calibrated with a large number of gage records in small, well-defined hydrologic regions.

Newton and Herrin, 1982, further recommend the following critera when evaluating existing flood frequency prediction procedures or when developing new procedures for a region.

1. Statistical regression methods with low standard errors of estimates should be used to develop the prediction equations and Bulletin 17B procedures applied for their calibration to flood frequency estimates.

2. Well-defined hydrologic regions should be used with the density of gages comparable to that of the USGS State regression equations.

3. Parameters used in the prediction equations must be uniquely defined and consistently measurable. Factors requiring user judgment should be avoided.

4. An application time of about 3 hours should be sufficient to estimate peak flows of specified frequency unless more complex analysis or watershed modeling is justified by the need for accuracy in the project.

# 6.0 DETERMINATION OF FLOOD HYDROGRAPHS

Often it is necessary to estimate the hydrograph or to develop a design hydrograph associated with a peak discharge. Methods presented in this section will permit the highway designer to develop these hydrographs. The section is divided basically into three parts. The first introduces the concept of the unit hydrograph and how it may be used to generate the design hydrograph for any duration storm; the second part presents methods for determining hydrographs when the requisite precipitation and surface runoff data are available; and the third part of this section discusses methods for developing synthetic hydrographs when insufficient or no data are available.

## 6.1 Unit Hydrographs

In Section 2.0 of this manual, it was shown that the rainfall-surface runoff relationship of a watershed is the result of the interaction of the hydrologic abstraction processes and the hydraulic conveyance of the primary and secondary drainage system. To accurately model this relationship mathematically and to predict the response of a watershed to any precipitation event is not totally possible at this time. There has been some success in this area through the use of sophisticated computer simulations but these require large amounts of data for calibration to be accurate. These techniques are outside the normal level of effort justified in typical highway drainage design. A more practical tool is necessary. Highway designers can use the techniques of unit hydrographs to approximate the rainfall runoff response of typical watersheds. These methods do not require as much data and are usually accurate enough for highway stream crossing design.

### 6.1.1 Assumptions

A hydrograph is simply a plot of discharge versus time. A runoff hydrograph is a plot of discharge due to direct runoff versus time. Since direct runoff results from excess rainfall, the runoff hydrograph is a plot of the response of a watershed to some rainfall event. If, for example, a rainfall event lasted for 1 hour, then the corresponding runoff hydrograph would be the response of the given watershed to a 1-hour storm. Figure 38 illustrates the runoff hydrograph from a rainfall of 1-hour duration.

Suppose that the same watershed was subjected to another storm that was the same in all respects except that it was twice as intense. The unit hydrograph technique assumes that the time base of the runoff hydrograph remains unchanged and that the ordinates are directly proportional to the amount of excess rainfall. In this particular case, the ordinates are twice as high as for the previous storm. This is illustrated in Figure 39.

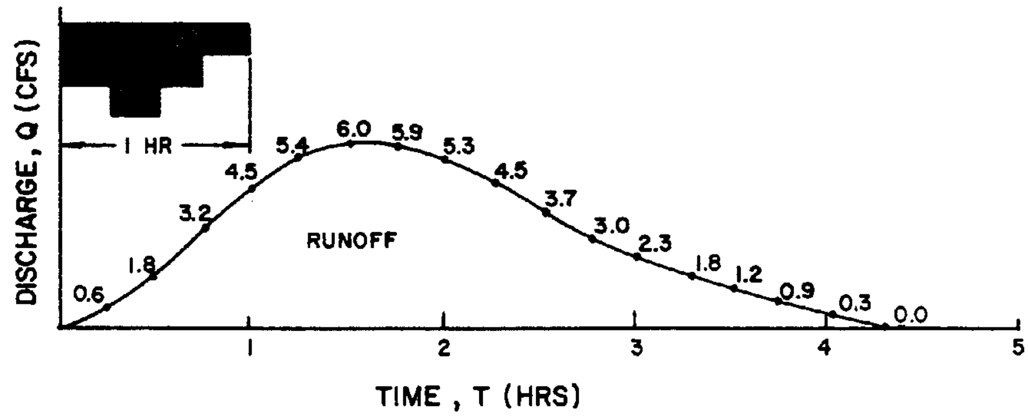



Figure 38. Runoff Hydrograph for 1-Hour Storm

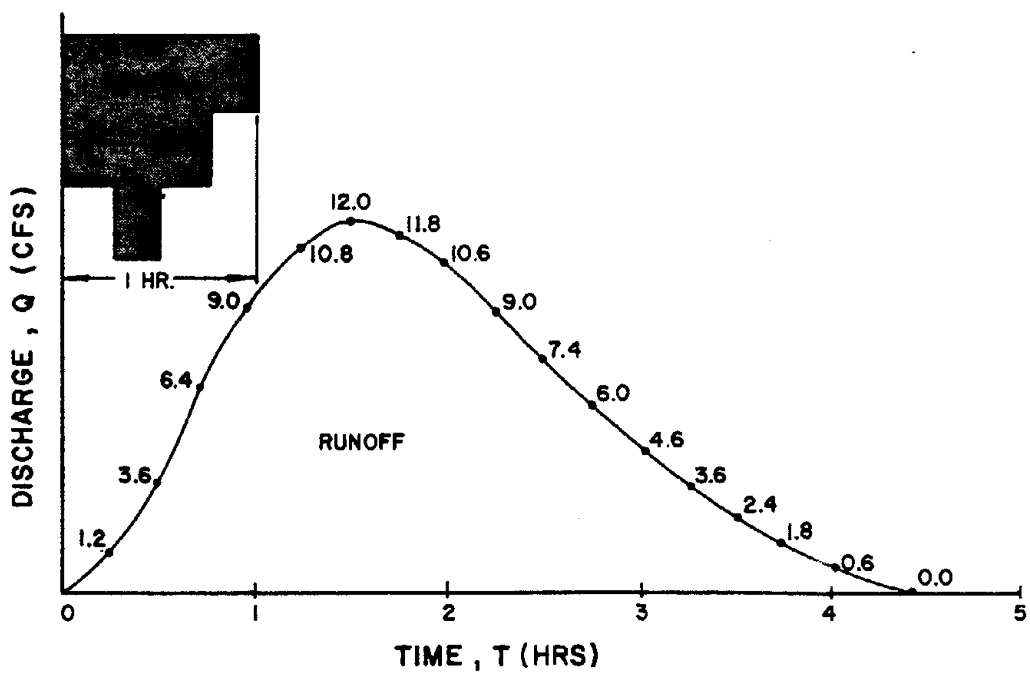



Figure 39. Runoff Hydrograph for 1-Hour Storm-Twice the Intensity

Now suppose if immediately after the 1-hour storm shown in Figure 38, another storm of exactly the same intensity and spatial distribution occurred. Unit hydrograph procedures assume also that the second runoff hydrograph is independent of antecedent conditions. It would be exactly the same as the first hydrograph and would be additive to the first except lagged 1 hour. The resulting hydrograph would be as illustrated in Figure 40.

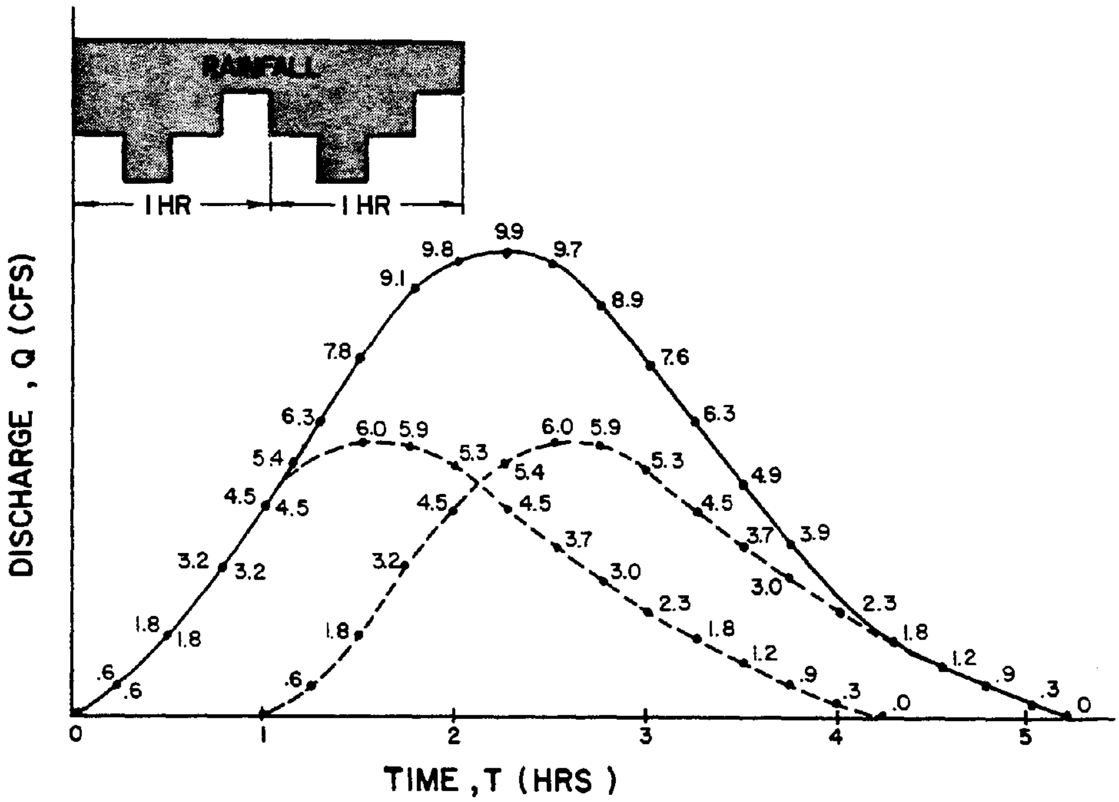



Figure 40. Runoff Hydrograph for Successive 1-Hour Storms

The above examples serve to illustrate the underlying assumptions applicable to unit hydrograph techniques.

### 6.1.2 Definition of Unit Hydrograph

A unit hydrograph is defined as the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution and which lasts for a unit duration of time. The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area.

It is to be noted that the characteristics of the unit hydrograph also depend on the duration of rainfall. In all probability, the unit hydrograph for a 1-hour storm will be quite different from the unit hydrograph for a 6-hour storm. The unit hydrograph is also dependent on the temporal and spatial distribution of the rainfall excess. In other words, two rainfall events with different distributions over the drainage area will give different hydrographs even if their respective durations are identical.

The key to applying unit hydrograph techniques in design problems is to select the correct rainfall event. The chosen storm must be representative of the temporal and spatial distribution of rainfall which is characteristic of storms resulting in peak discharges of the magnitudes and frequency selected for design. The selection of design storms is treated in a subsequent part of this section.

### 6.1.3 Construction of Unit Hydrographs from Gaged Data

Unit hydrographs are either determined from gaged data or they are derived from empirically based synthetic unit hydrograph procedures. This section deals with the derivation of unit hydrographs from data. It would be fortunate indeed if there were a continuous streamflow gage exactly at or near the site where there is need to design a highway crossing. This, however, is seldom the case. The unit hydrograph approach would, therefore seem to have limited application, but unit hydrographs can be transposed within hydrologically similar regions using techniques discussed later. A unit hydrograph can be developed at a location where the necessary data are available and then transposed to the design site, so long as the distances are not too great and the watersheds are similar.

The first step in deriving a unit hydrograph is the collection of the necessary data. Data collection and sources were discussed in Section 3.0. It would be beneficial to keep a directory of all recording stream gages and associated precipitation stations within a region. This would facilitate data collection and streamline the process when a hydrograph design was required.

The data needed for unit hydrograph development are precipitation and continuous streamflow records for storms which are of a recurrence interval close to the anticipated design recurrence interval. It is not reasonable to expect that the response of a watershed will be the same for a 2-year storm as for a 50-year storm. Ideally, the hydrograph should have a single peak and the precipitation should be isolated and uniform in time and space over the watershed. In addition, the entire basin should be contributing and the storm should be sufficiently large so that the runoff hydrograph is well defined. If the deviation from these criteria is too extreme, it might be better to resort to a synthetic unit hydrograph procedure. Assuming that the data are usable, then the following procedure is used to derive a unit hydrograph.

6.1.3.1 Base Flow Separation

The first step in developing a unit hydrograph is to separate base flow and determine the direct runoff hydrograph. Figure 41 illustrates a typical record obtained from a continuous recording gage. Prior to the occurrence of the storm, the flow in the stream is determined by groundwater depletion and is referred to as base flow. After the passage of the flood, the discharge in the stream returns to the base flow. The base flow is assumed to be unrelated to the storm runoff and, therefore, must be eliminated in order to determine the direct runoff hydrograph.

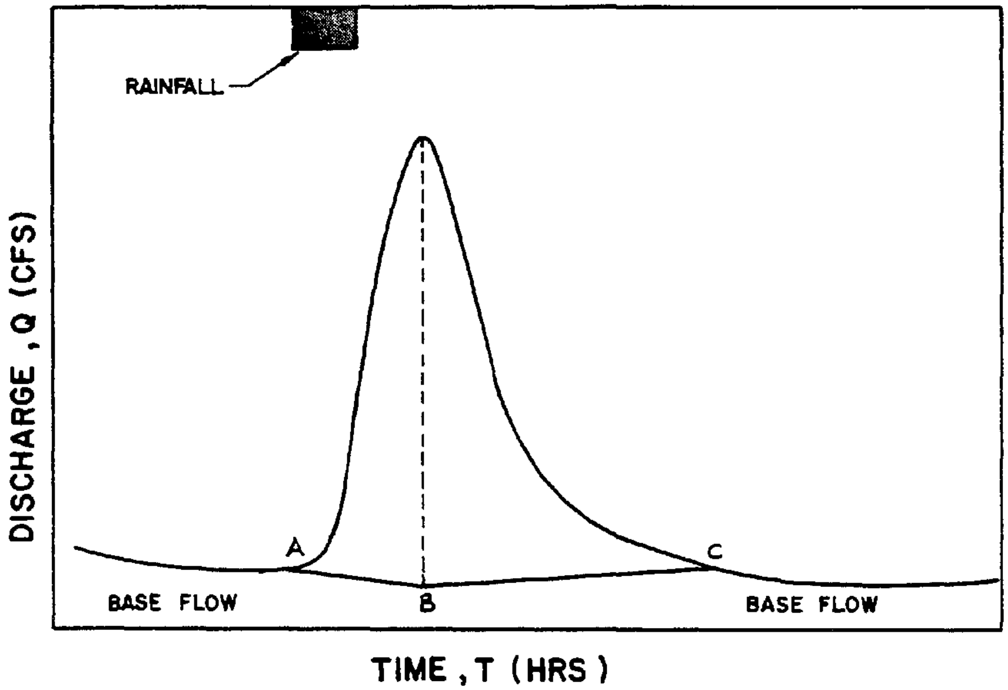



Figure 41. Base Flow Separation

There are a number of techniques that have been proposed for separating the base flow from the flood hydrograph. Since the base flow is usually small in relation to the flood discharge, the simple straight line separation described below is adequate for most highway design purposes.

A straight line is drawn from the beginning of the rising portion of the hydrograph to a point directly below the peak of the hydrograph. The slope of this line is the same as the slope of the base flow curve prior to the rise of the hydrograph. This is line AB in Figure 41. A second straight line is drawn from point B to point C on the recession limb of the hydrograph, Figure 41, where the baseflow is equal to that which existed at point A. This procedure is applicable where groundwater recharge and possible subsequent increases in baseflow are not significant. This would commonly be the case for smaller watersheds and intense storms. For larger watersheds or for long duration storms, some judgment may be required for drawing line BC.

6.1.3.2 Direct Runoff Volume

The direct runoff hydrograph is obtained by subtracting the base flow from the flood hydrograph. From the direct runoff hydrograph it is possible to determine the total volume of direct runoff. This is simply the area under the hydrograph. This volume is next converted to an equivalent depth of uniform rainfall over the entire drainage basin (the area of the drainage basin must be known) as illustrated below:

The direct runoff hydrograph ordinates at 15 minute intervals are tabulated in the first two columns of Table 32 for a drainage basin with an area of 0.9 sq mi (576 acres or 2.3 sq km).

The volume within each time increment of the direct runoff hydrograph is determined by taking the average discharge for the time increment and multiplying that discharge by the time per increment. The total volume is obtained by adding the volumes for all the time increments.

For the first time increment the average discharge is

$$\frac{0+6}{2} = 3 \text{ CFS } (0.08 \text{ CMS})$$

The incremental volume is

$$\frac{3 \text{ cu ft}}{\text{sec}} \times 15 \text{ min} \times \frac{60 \text{ sec}}{\text{min}} = 2700 \text{ cu ft} \quad (76.5 \text{ cu m})$$

This process is repeated for the entire hydrograph as shown in Table 32.

6.1.3.3 Determination of Unit Hydrograph

The ordinates of the unit hydrograph are determined by dividing the ordinates of the direct runoff hydrograph by the volume of runoff (in inches) from the drainage area. This computation is also shown in Table 32 together with a check on the volume of runoff. The total volume of runoff under the unit hydrograph should equal 1.0 inch. If not, some minor adjustments to the unit hydrograph ordinates should be made and the volume re-computed. Both the direct runoff and unit hydrographs are plotted in Figure 42.

Table 32. Computation of Direct Runoff and Unit Hydrograph Volumes

| Time | Direct Runoff Discharge CFS | Average Direct Runoff Discharge CFS | Incremental Direct Runoff Volume cu ft | Average Unit Hydrograph Discharge CFS | Average Unit Hydrograph Incremental Volume cu ft |
|---|---|---|---|---|---|
| 1:45 p.m. | 0.0 | 3.0 | 2,700 | 13.8 | 12,420 |
| 2:00 p.m. | 6.0 | 12.0 | 10,800 | 55.3 | 49,770 |
| 2:15 p.m. | 18.0 | 25.0 | 22,500 | 115.2 | 103,680 |
| 2:30 p.m. | 32.0 | 38.0 | 34,200 | 175.1 | 157,590 |
| 2:45 p.m. | 44.0 | 49.0 | 44,100 | 225.8 | 203,220 |
| 3:00 p.m. | 54.0 | 57.0 | 51,300 | 262.7 | 236,430 |
| 3:15 p.m. | 60.0 | 59.5 | 53,550 | 274.2 | 246,780 |
| 3:30 p.m. | 59.0 | 56.0 | 50,400 | 258.1 | 232,290 |
| 3:45 p.m. | 53.0 | 49.0 | 44,100 | 225.8 | 203,220 |
| 4:00 p.m. | 45.0 | 41.0 | 36,900 | 188.9 | 170,010 |
| 4:15 p.m. | 37.0 | 33.5 | 30,150 | 154.4 | 138,960 |
| 4:30 p.m. | 30.0 | 26.5 | 23,850 | 122.1 | 109,890 |
| 4:45 p.m. | 23.0 | 20.5 | 18,450 | 94.5 | 85,050 |
| 5:00 p.m. | 18.0 | 15.0 | 13,500 | 69.1 | 62,190 |
| 5:15 p.m. | 12.0 | 10.5 | 9,450 | 48.4 | 43,560 |
| 5:30 p.m. | 9.0 | 6.0 | 5,400 | 27.6 | 24,840 |
| 5:45 p.m. | 3.0 | 1.5 | 1,350 | 6.9 | 6,210 |
| 6:00 p.m. | 0.0 | TOTAL VOLUME = | 452,700 $ft^3$ | = | 2,086,110 $ft^3$ |

Converting the total volume of direct runoff to an equivalent depth of water over the entire drainage area gives:

$$\frac{452{,}700\,ft^3}{576\text{ acres}} \times \frac{1\text{ acre}}{43{,}560\,ft^2} \times \frac{12\text{ in}}{1\text{ ft}} = 0.217\text{ in}$$

Now checking the total volume of runoff from the unit hydrograph gives:

$$\frac{2{,}086{,}110\,ft^3}{576\text{ acres}} \times \frac{1\text{ acre}}{43{,}560\,ft^2} \times \frac{12\text{ in}}{1\text{ ft}} = 0.998\text{ in}$$

The error in the unit hydrograph volume is 0.2 percent which is acceptable for use in highway design.

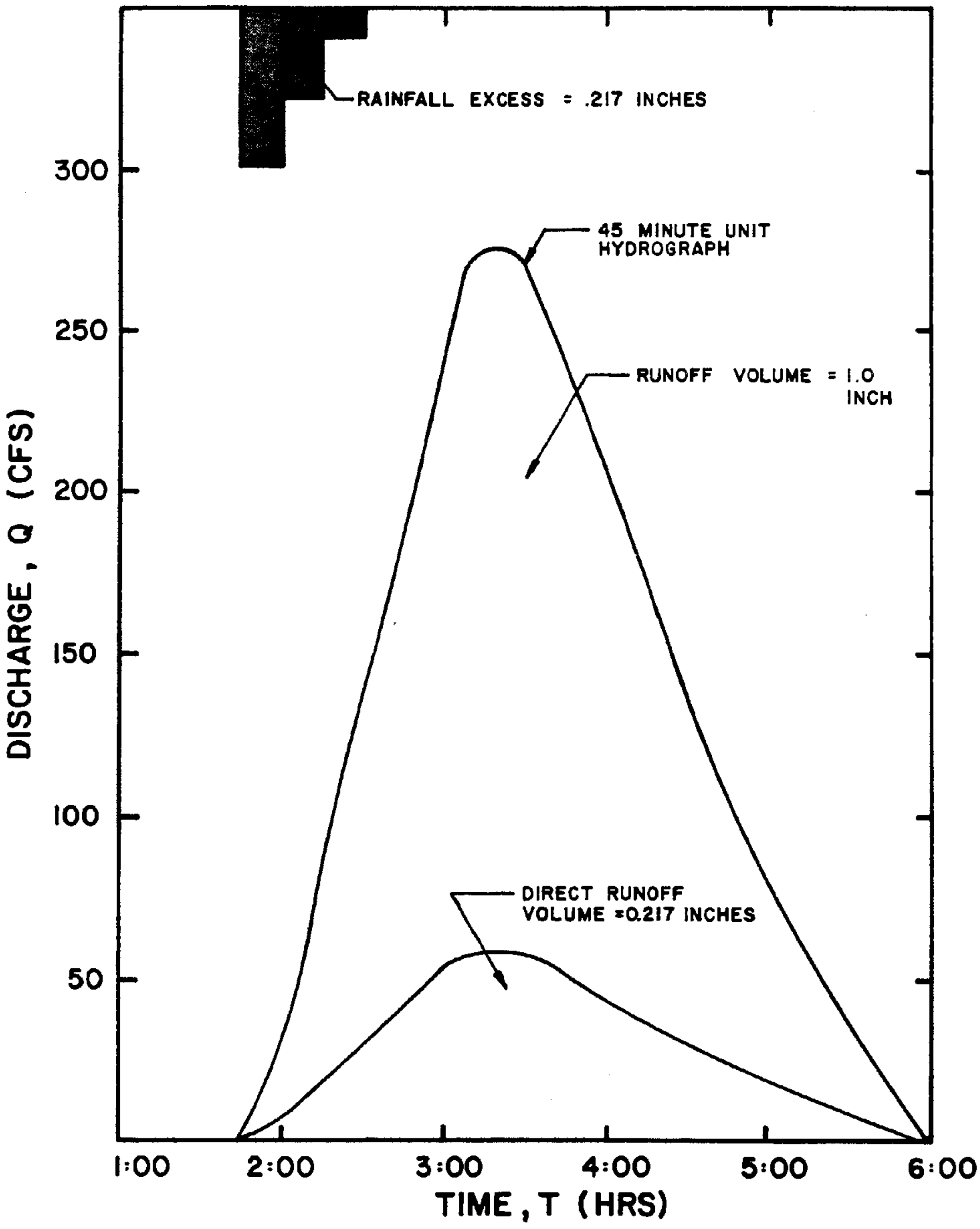



Figure 42. Direct Runoff and Unit Hydrographs

#### 6.1.3.4 Determination of Duration of Excess Rainfall

Next the precipitation records for the storm which produced the direct runoff hydrograph are analyzed to determine the duration of excess rainfall. The designer must be guided in this effort by an understanding of the type and relative magnitudes of the abstractions which occur before rainfall runs off a watershed as discussed in Section 2.0. The designer must also appreciate that the precipitation records are a sample of the actual precipitation which produced the runoff event and that variations in areal extent and time distribution of rainfall might have occurred which are not represented in the rainfall data.

Because of the complexity of the rainfall runoff process and the limited data which are usually available, a simple version of the $\Phi$ (Phi) Index method is used to determine the duration of rainfall excess. If more data are available, especially concerning small scale time distributions of rainfall and relative infiltration capacities of the various soil types which exist in the watershed, then more sophisticated techniques are certainly preferred. These are not discussed in this manual but are treated in detail in standard hydrology texts.

For the direct runoff hydrograph illustrated above the corresponding precipitation records are:

| Time | Rainfall Intensity | Depth of Rain |
|---|---|---|
| 1:30 p.m. | 0.4 inches/hour | 0.4 in/hr X .25 hr = 0.10 in |
| 1:45 p.m. | 0.6 inches/hour | 0.6 in/hr X .25 hr = 0.15 in |
| 2:00 p.m. | 0.4 inches/hour | 0.4 in/hr X .25 hr = 0.10 in |
| 2:15 p.m. | 0.2 inches/hour | 0.2 in/hr X .25 hr = 0.05 in |
| | | 0.40 in |

The total depth of rainfall is 0.4 inches. Since the depth of direct runoff was 0.217 inches, 0.183 inches of rain were lost due to a variety of hydrologic abstractions. The problem now is to determine a reasonable pattern of rainfall excess in a simple and straightforward manner.

The hyetograph of the precipitation is shown in Figure 43 below:

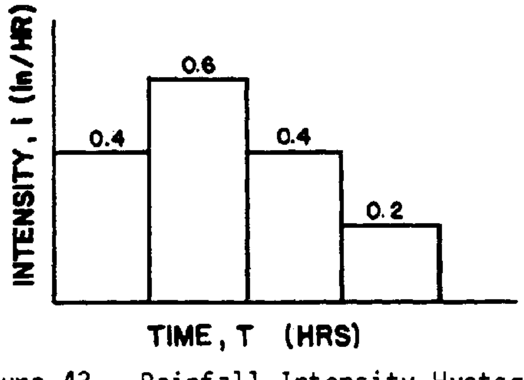



Figure 43. Rainfall Intensity Hyetograph

Notice that the rainfall began at 1:30 but that the corresponding runoff does not begin until 1:45 p.m. It is therefore assumed that all of the rain falling in the first 15 minute period was lost due to initial abstractions and infiltration. The remaining volume of rainfall is (0.4 - 0.1 inches) or 0.3 inches, which is still greater than the 0.217 inches which ran off. Therefore there are additional losses to account for. This is done by applying the $\Phi$ index method.

The $\Phi$ method assumes that there is a constant loss rate which will result in an excess rainfall depth equal to the direct runoff depth. The problem is to solve for this constant loss rate. For the rainstorm being used in the above example problem, it is possible now to solve for the $\Phi$ value. This is illustrated in Figure 44 below:

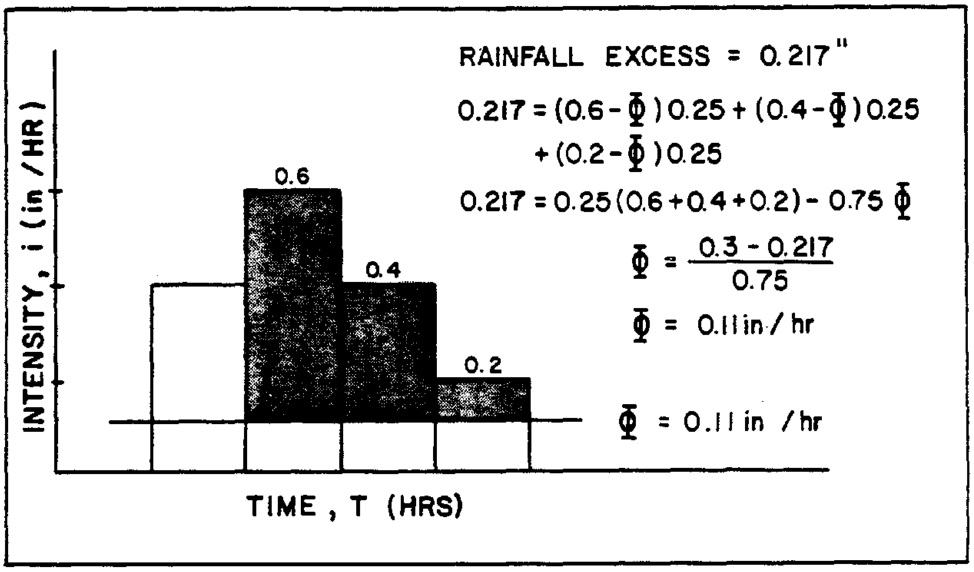



Figure 44. Determination of Excess Rainfall by $\Phi$ Index Method

The $\Phi$ index value is computed to be 0.11 inches/hour. The shaded area in Figure 44 defines the duration and intensity pattern of the excess rainfall and its volume is 0.217 inches. This now completely defines the 45-minute unit hydrograph and the direct runoff hydrograph which have total volumes of 1.0 inches and 0.217 inches, respectively, and which are distributed in time as shown in Figure 43.

### 6.1.4 Complex Storms

The unit hydrograph provides a convenient method for developing hydrographs for other rainstorms provided they are of the same unit duration and have spatial and temporal patterns similar to the one used to develop the unit hydrograph. A new flood hydrograph is determined by simply multiplying the unit hydrograph ordinates by the volume of surface runoff (in inches) from the new storm.

This might be useful if all the storms for which design hydrographs are developed are very similar. Unfortunately, this is rarely the case. There is a need for a more useful tool, one which can be applied to a different pattern of rainfall excess. What is needed is a unit hydrograph for a single time duration.

#### 6.1.4.1 Compounding Unit Hydrographs

From the assumptions that the distribution of runoff is independent of antecedent conditions and that the instantaneous flow is directly proportional to the amount of runoff, it is possible to develop the unit hydrograph for a single time duration.

Such a unit hydrograph can be derived from the direct runoff hydrograph in the example above. The direct runoff hydrograph is the result of a rainfall excess which consists of three equal duration periods of uniform excess rainfall of 0.49 inches per hour, 0.29 inches per hour and 0.09 inches per hour, Figure 44. If it is assumed that the direct runoff hydrograph is the composite of three separate hydrographs, each produced by one of the periods of excess rainfall, then it is possible to work backwards and derive a 15 minute unit hydrograph for a uniform excess rainfall intensity of 4 inches per hour (this would result in a direct runoff volume of 1 inch). These calculations are illustrated by the example below and the resulting unit hydrograph is plotted in Figure 45.

The following symbols are used:

Q(M) = Direct Runoff Hydrograph Ordinate (CFS)

R(M) = Excess Rainfall Intensity (inches/hour)

U(M) = 15 Minute Unit Hydrograph Ordinate (CFS)

For each value of the direct runoff hydrograph determined from the gage data, an equation can be written as shown below.

```
 Q(1) = R(1) X U(1) = 6 CFS = 0.49 X U(1)
 Q(2) = R(1) X U(2) + R(2) X U(1) = 18 CFS = 0.49 X U(2) + 0.29 X U(1)
 Q(3) = R(1) X U(3) + R(2) X U(2) + R(3) X U(1) = 32 CFS = 0.49 X U(3) +
        0.29 X U(2) + 0.09 X U(1)
 Q(4) = R(1) X U(4) + R(2) X U(3) + R(3) X U(2) = 44 CFS = 0.49 X U(4) +
        0.29 X U(3) + 0.09 X U(3)
 Q(5) = R(1) X U(5) + R(2) X U(4) + R(3) X U(3) = 54 CFS = 0.49 X U(5) +
        0.29 X U(4) + 0.09 X U(3)
 Q(6) =                  = 0.49 X U(6) + 0.29 X U(5) + 0.09 X U(4)
 Q(7) =                  = 0.49 X U(7) + 0.29 X U(6) + 0.09 X U(5)
 Q(8) =                  = 0.49 X U(8) + 0.29 X U(7) + 0.09 X U(6)
 Q(9) =                  = 0.49 X U(9) + 0.29 X U(8) + 0.09 X U(7)
Q(10) =                  = 0.49 X U(10) + 0.29 X U(9) + 0.09 X U(8)
Q(11) =                  = 0.49 X U(11) + 0.29 X U(10) + 0.09 X U(9)
Q(12) =                  = 0.49 X U(12) + 0.29 X U(11) + 0.09 X U(10)
```

Q(13) = = 0.49 X U(13) + 0.29 X U(12) + 0.09 X U(11)
Q(14) = R(1) X U(14) + R(2) X U(13) + R(3) X U(12) = 12 CFS = 0.49 U(14) + 0.29 X U(13) + 0.09 X U(12)
Q(15) = R(2) X U(14) + R(3) X U(13) = 9 CFS = 0.29 U(14) + 0.09 U(13)
Q(16) = R(3) X U(14) = 3 CFS = 0.09 X U(14)
Q(17) = 0

Starting at the top, each equation is solved in turn for a single unknown, i.e., the value of the unit hydrograph ordinate U(M).

The values of the 15-minute unit hydrograph ordinates obtained by solving the equations above are:

| | |
|---|---|
| U(1) = 12.2 CFS | U(8) = 56.2 CFS |
| U(2) = 29.5 CFS | U(9) = 46.5 CFS |
| U(3) = 45.6 CFS | U(10) = 37.6 CFS |
| U(4) = 57.4 CFS | U(11) = 30.4 CFS |
| U(5) = 67.8 CFS | U(12) = 22.0 CFS |
| U(6) = 71.7 CFS | U(13) = 18.1 CFS |
| U(7) = 65.5 CFS | U(14) = 9.7 CFS |

The unit hydrograph is plotted in Figure 45 together with the direct runoff hydrographs for each 15-minute rainfall duration.

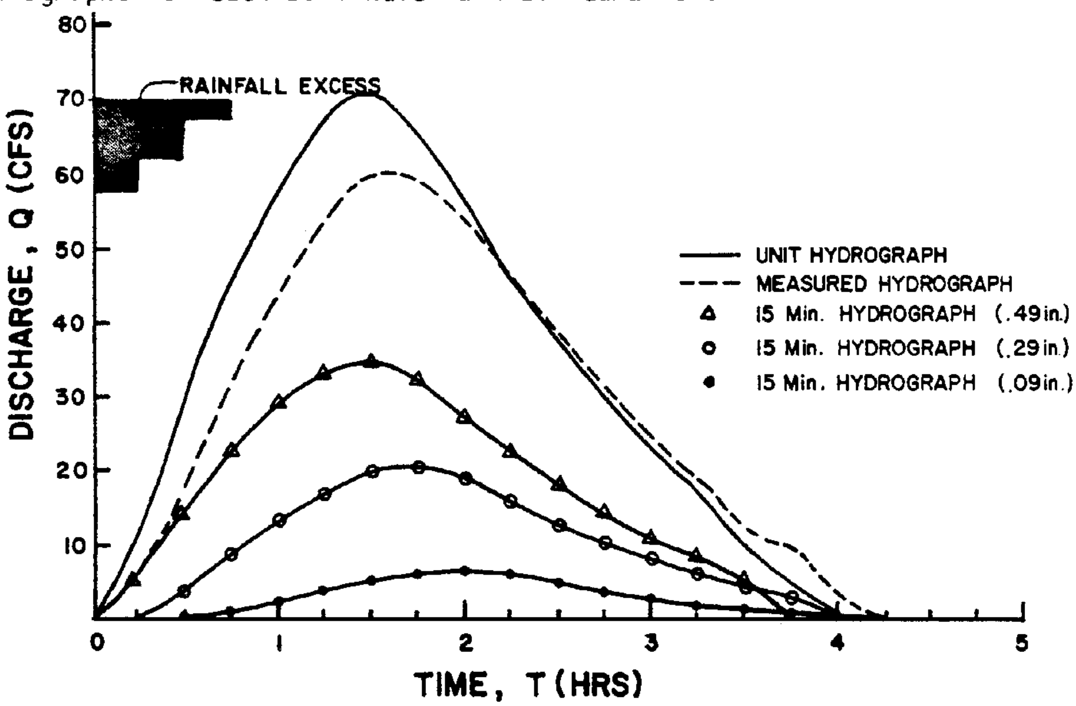
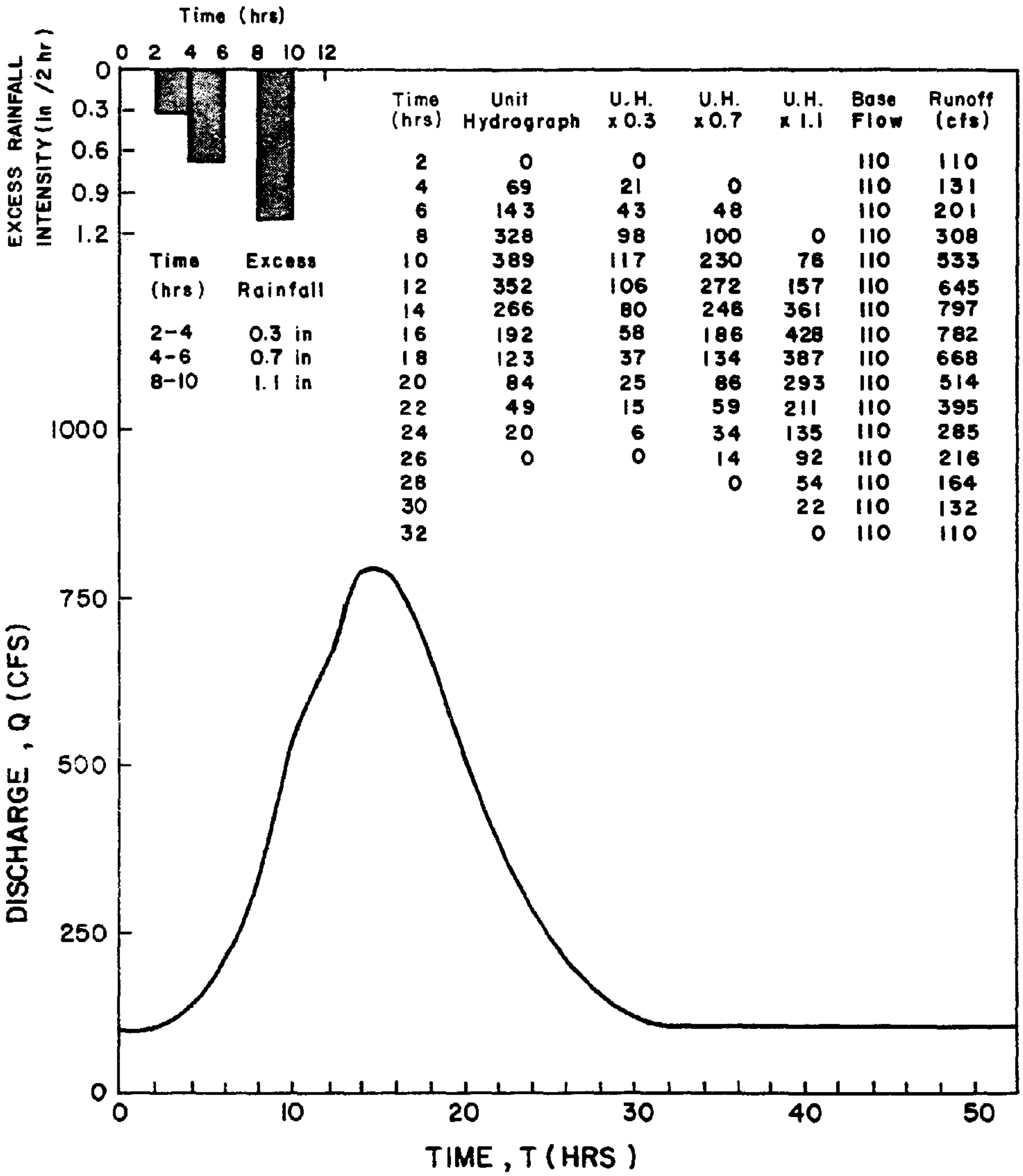



Figure 45. Unit Hydrograph from Compounded Direct Runoff Hydrographs

Another example of compounding hydrographs is given by Sanders, 1980. In this problem the unit hydrograph ordinates have been determined for a 2-hour unit duration, and it is desired to compute the flood hydrograph for a complex storm over a 10-hour period. The excess rainfall, all calculations and the resulting flood hydrograph are illustrated in Figure 46. (Note: The




| Time (hrs) | Excess Rainfall |
|---|---|
| 2-4 | 0.3 in |
| 4-6 | 0.7 in |
| 8-10 | 1.1 in |

| Time (hrs) | Unit Hydrograph | U.H. x 0.3 | U.H. x 0.7 | U.H. x 1.1 | Base Flow | Runoff (cfs) |
|---|---|---|---|---|---|---|
| 2 | 0 | 0 | | | 110 | 110 |
| 4 | 69 | 21 | 0 | | 110 | 131 |
| 6 | 143 | 43 | 48 | | 110 | 201 |
| 8 | 328 | 98 | 100 | 0 | 110 | 308 |
| 10 | 389 | 117 | 230 | 76 | 110 | 533 |
| 12 | 352 | 106 | 272 | 157 | 110 | 645 |
| 14 | 266 | 80 | 246 | 361 | 110 | 797 |
| 16 | 192 | 58 | 186 | 428 | 110 | 782 |
| 18 | 123 | 37 | 134 | 387 | 110 | 668 |
| 20 | 84 | 25 | 86 | 293 | 110 | 514 |
| 22 | 49 | 15 | 59 | 211 | 110 | 395 |
| 24 | 20 | 6 | 34 | 135 | 110 | 285 |
| 26 | 0 | 0 | 14 | 92 | 110 | 216 |
| 28 | | | 0 | 54 | 110 | 164 |
| 30 | | | | 22 | 110 | 132 |
| 32 | | | | 0 | 110 | 110 |

Figure 46. Complex Storm Hydrograph

base flow which was initially separated out before determining the unit hydrograph, is added back to the direct runoff in order to determine the flood hydrograph.)

6.1.4.2 Varying Durations

Again, based on the unit hydrograph assumptions, it is possible to transform a unit hydrograph of specified duration into one with a different duration. There are basically two methods to accomplish this transformation. The first applies to developing a longer duration unit hydrograph from a shorter duration where the longer duration is an equal or near equal multiple of the shorter duration.

Suppose it is desired to find a 6-hour unit hydrograph from an existing 3-hour unit hydrograph (1 inch of excess rainfall in 3 hours). Assuming independence of antecedent conditions, a second 3-hour unit graph is lagged or displaced 3 hours from the first as illustrated in Figure 47. The ordinates are then added which yields 2 inches of runoff in 6 hours. Dividing these ordinates by 2 gives the 6-hour unit hydrograph also shown in Figure 47.

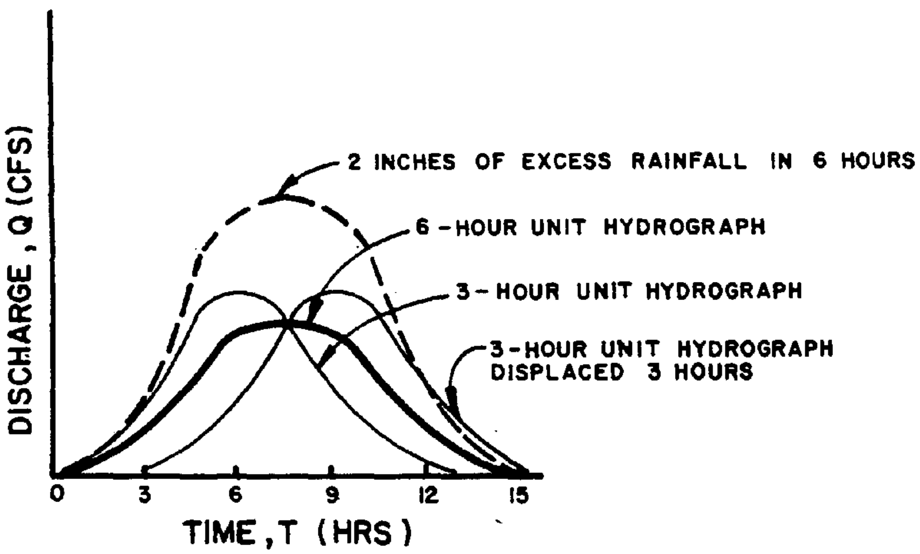



Figure 47. Lagging Unit Hydrographs

To change the unit hydrograph from a longer duration to a shorter duration or to any duration which is not a multiple of the shorter duration it is necessary to develop the "S" Curve (Summation Curve). The "S" Curve is the summation of an infinite number of unit hydrographs of specified duration each lagged from the preceding one by the duration of rainfall excess as shown in Figure 48. The S-Curve approaches a constant value of the discharge equal to (1-inch) x (drainage area)/unit duration in consistent units, so practically it is necessary to include only enough lagged unit hydrographs to define the "S" Curve up to this level.

The unit hydrograph for a new specified duration is obtained by lagging the "S" Curve by the new duration, subtracting the two "S" Curves from one another and multiplying the resulting hydrograph ordinates by the ratio of the duration of the unit hydrograph used to construct the "S" curve to the duration of the unit hydrograph being developed. For example, if a 3-hour unit graph is to be developed from a 6-hour unit hydrograph, the ordinates are multiplied by two (2) to obtain a volume equal to 1 inch. Similarly, in going from 6 hours to 15 hours, the multiplier is 6/15 or 2/5.

Using Figure 48, Sanders, 1980, gives an example of the "S" Curve computations in which a 2-hour unit hydrograph is used to determine the 4-hour unit hydrograph. These computations are summarized in Table 33.

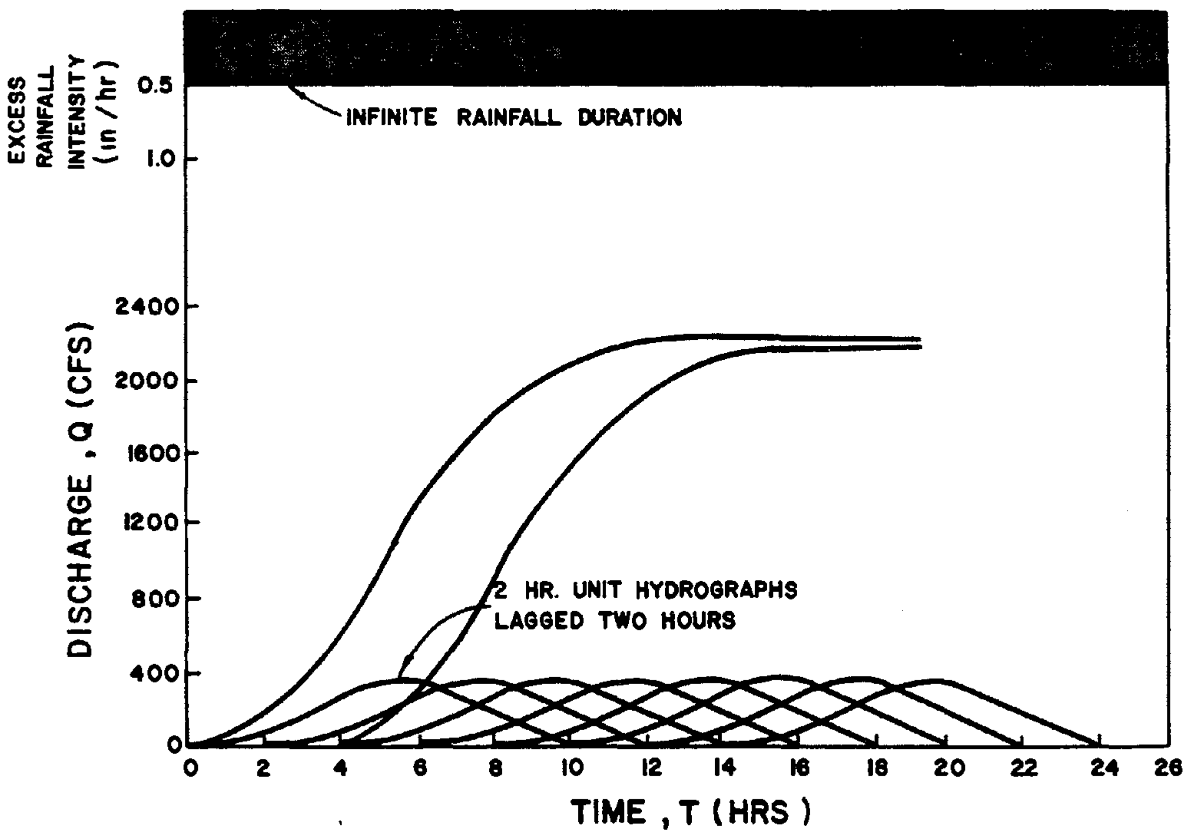



Figure 48. Graphical Illustration of the S-Curve Construction

### 6.1.5 Unit Hydrograph Limitations

Because of the assumptions made in the development of unit hydrograph procedures, there are several limitations and sources of error with which the designer should be familiar. Uniformity of rainfall intensity and duration over the drainage basin is a requirement that is seldom met. For this reason

Table 33. S-Curve Determined from a 2-Hour Unit Hydrograph to Estimate a 4-Hour Unit Hydrograph.

| Time Hrs | 2-Hr Unit Hydrograph | S-Curve | Lagged S-Curve | 4-Hr Hydrograph | 4-Hr Unit Hydrograph |
|---|---|---|---|---|---|
| 0 | 0 | 0 | -- | 0 | 0 |
| 2 | 69 | 69 | -- | 69 | 34 |
| 4 | 143 | 212 | 0 | 212 | 106 |
| 6 | 328 | 540 | 69 | 471 | 235 |
| 8 | 389 | 929 | 212 | 717 | 358 |
| 10 | 352 | 1281 | 540 | 741 | 375 |
| 12 | 266 | 1547 | 929 | 618 | 309 |
| 14 | 192 | 1739 | 1281 | 458 | 229 |
| 16 | 123 | 1862 | 1547 | 315 | 158 |
| 18 | 84 | 1946 | 1739 | 207 | 103 |
| 20 | 49 | 1995 | 1862 | 133 | 66 |
| 22 | 20 | 2015 | 1946 | 69 | 34 |
| 24 | 0 | *2015 | 1995 | 20 | 10 |
| 26 | 0 | *2015 | 2015 | 0 | 0 |

* adjusted values
from Sanders, 1980

it is best to take large storms covering a major portion of the drainage area. If the basin is only partially covered, a routing problem may be involved. To minimize the effects of non-uniform distribution of rainfall, an average unit hydrograph of a specified unit duration might be considered from several major storms. This average unit hydrograph should be developed from the average peak flow, and time to peak, with the shape of the unit hydrograph adjusted to a volume of 1-inch of runoff.

The lack of stations with recording rain gages makes it very difficult to obtain accurate rainfall distribution data. Even bucket-type gages may have limitations because they are read only periodically, e.g. every 24 hours. Thus, a single reading in a 24-hour period would introduce serious error in the rainfall intensity if in fact all the precipitation occurred in the first 6 hours. Inadequate rainfall intensity data will introduce errors in both the peak flow and time to peak of the unit hydrograph.

Storm movement is still another consideration in the development of unit hydrographs, especially for basins that are relatively narrow and long. Generally, storms moving down the basin will result in hydrographs with higher peak flows and longer times to peak than comparable storms moving up the basin. In order to overcome some of these limitations, unit hydrograph development should be limited to drainage areas less than 1000 square miles and should not under any circumstances be used when the area is in excess of 3000 square miles.

Finally, it should be remembered that the unit hydrograph will be no more accurate than the data from which it is developed. In contrast to frequency analysis where documented historical peak flows are estimated and included in the analysis with little error, the reliability of hydrograph analyses is directly impacted by the accuracy of the data due to lack of continuous records or gage malfunction.

### 6.2 Synthetic Unit Hydrographs for Basins Without Data

The United States covers a broad spectrum of geographical and climatic regimes. Consequently, no one nationwide synthetic unit hydrograph method is applicable throughout the country. Therefore, a number of different synthetic unit hydrograph procedures have evolved. Two of the most widely used are the Snyder method and the Soil Conservation Service method.

#### 6.2.1 Snyder Synthetic Hydrograph

This method developed in 1938 has been used extensively by the Corps of Engineers and provides a means of generating a synthetic unit hydrograph. In the Snyder method, two empirically defined terms, $C_t$ and $C_p$, and the physiographic characteristics of the drainage basin are used to determine a unit hydrograph. The entire time distribution of the unit hydrograph is not explicitly determined using this method. Certain key parameters of the unit hydrograph are evaluated and from these a characteristic unit hydrograph is constructed. The key parameters which are explicitly calculated are the lag time, the unit hydrograph duration, the peak discharge and the hydrograph time widths at 50 percent and 75 percent of the peak discharge. With these points a characteristic unit hydrograph is sketched. The volume of this hydrograph is then checked to ensure it equals 1 inch of runoff. If it does not, it is adjusted accordingly. A typical Snyder hydrograph is shown in Figure 49 below.

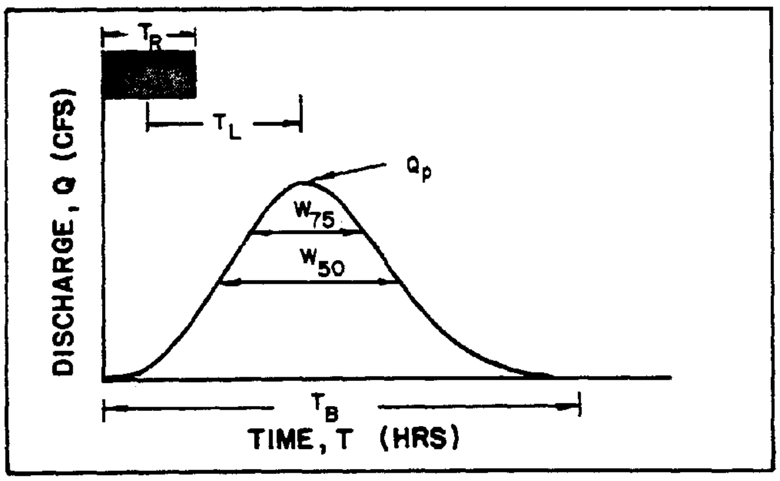



Figure 49. Snyder Synthetic Hydrograph Definitions

A step-by-step procedure to develop the Snyder unit hydrograph is presented as follows:

1. Data Collection and Determination of Physiographic Constants

Snyder developed his method using data for watersheds in the Appalachian Highlands and consequently the values derived for the constants $C_t$ and $C_p$ are characteristic of this area of the country. However, the general method has been successfully applied throughout the country by appropriate modification of these empirical constants. Values for $C_t$ and $C_p$ need to be determined for the watershed under consideration. These can be obtained by analyzing unit hydrographs derived for gaged streams in the same general area. Another source of information is the Corps of Engineers, District Offices, which are listed in Appendix C. $C_t$ is a coefficient which represents the variation of unit hydrograph lag time with watershed slopes and storage. In his Appalachian Highlands study, Snyder found $C_t$ to vary from 1.8 to 2.2. Further studies have shown that extreme values of $C_t$ vary from 0.4 in Southern California to 8.0 in the Eastern Gulf of Mexico. $C_p$ is a coefficient which represents the variation of unit hydrograph peak discharge with watershed slope, storage, lag time and effective area. Values of $C_p$ range between 0.4 and 0.94.

In addition to these empirical coefficients, the watershed area, A, in sq mi, the length along the main channel from the outlet to the divide, L in mi, and the length along the main channel to a point opposite the watershed centroid, $L_{ca}$ in mi, need to be determined from available topographic maps.

2. Determination of Lag Time

The next step is to determine the lag time, $T_L$, of the unit hydrograph. The lag time is the time from the centroid of the excess rainfall to the hydrograph peak. Snyder derived the following empirical equation for lag time

$$T_L = C_t (LL_{ca})^{0.3} \qquad (6\text{-}1)$$

where $T_L$ is the lag time in hours, $C_t$ is the empirical coefficient defined above, L is the length along main channel from outlet to divide in miles, and $L_{ca}$ is the length along main channel from outlet to a point opposite the watershed centroid in miles.

3. Determine Unit Hydrograph Duration

The relationship developed by Snyder for the duration of the excess rainfall, $T_R$ in hours, is a function of the lag time computed above, namely

$$T_R = \frac{T_L}{5.5} \quad (6\text{-}2)$$

Equation (6-2) always results in an initial value of $T_R$ of $T_L/5.5$. However, a relationship has been developed to adjust the computed lag time for other durations. This is necessary because the equation above results in inconvenient values of unit hydrograph duration. The adjustment relationship is

$$T_{L(adj.)} = T_L + 0.25(T_R' - T_R) \quad (6\text{-}3)$$

where $T_{L(adj.)}$ is the adjusted lag time for the new duration in hrs, $T_L$ is the original lag time as computed above in hrs, $T_R$ is the original duration (i.e. $T_L/5.5$) in hrs and $T_R'$ is the desired duration in hrs.

For example: If the originally computed lag time, $T_L$, was 12.5 hours, then the corresponding unit hydrograph duration would be (12.5/5.5) or 2.3 hours. It would be more convenient to have a duration of 2.0 hours so the lag time is adjusted as follows

$$\begin{aligned} T_{L(adj.)} &= T_L + 0.25(T_R' - T_R) \\ &= 12.5 + 0.25(2.0 - 2.3) \\ T_{L(adj.)} &= 12.43 \text{ HRS} \end{aligned}$$

An alternative procedure would be to use the S curve technique (Section 6.1.4.2), but the above procedure is much simpler.

4. Determine Peak Discharge

The peak discharge for the unit hydrograph is determined from the equation below

$$Q_p = \frac{640\, C_p A}{T_{L(adj.)}} \quad (6\text{-}4)$$

where $Q_p$ is the peak discharge in CFS, $C_p$ is the empirical coefficient defined above, and A is the watershed area in sq mi.

5. Determine Time Base of Unit Hydrograph

The time base, $T_B$, of the unit hydrograph was determined by Snyder to be approximately equal to

$$T_B = 3 + \frac{T_{L(adj.)}}{8} \qquad (6\text{-}5)$$

where $T_B$ is the time of the synthetic unit hydrograph in days. This relationship, while reasonable for larger watersheds, may not be applicable for smaller watersheds. A more realistic value for smaller watersheds, is to use 3 to 5 times the time to peak as a base for the unit hydrograph. The time to peak is the time from the beginning of the rising limb of the hydrograph to the peak.

6. Estimate $W_{50}$ and $W_{75}$

The time widths of the unit hydrograph at discharges equal to 50 percent and 75 percent of the peak discharges, $W_{50}$ and $W_{75}$ respectively, have been found to be approximated by the following equations

$$W_{50}(HR) = 735\left(\frac{Q_P}{A}\right)^{-1.075} \qquad (6\text{-}6)$$

and

$$W_{75}(HR) = 434\left(\frac{Q_P}{A}\right)^{-1.075} \qquad (6\text{-}7)$$

7. Construct Unit Hydrograph

Using the values computed in the previous steps, the unit hydrograph can now be sketched, remembering that the total volume of runoff must equal 1 inch. A rule of thumb to assist in sketching the unit hydrograph is that the $W_{50}$ and $W_{75}$ time widths should be apportioned with one third to the left of the peak and two thirds to the right of the peak.

The development of the Snyder unit hydrograph is illustrated by the example below.

A synthetic unit hydrograph is to be constructed for a watershed of 875 sq mi, where L is measured to be 83 mi and $L_{ca}$ is 40.6 mi. For this region, average values of $C_t$ = 1.32 and $C_p$ = 0.63 have been found to apply.

$$T_L = C_t (LL_{ca})^{0.3} = 1.32(83 \times 40.6)^{0.3} = 15.1 \text{ HRS}$$

$$T_R = \frac{T_L}{5.5} = 2.75 \text{ HRS}$$

A 3-hour unit hydrograph is desired.

$$T_{L\,(adj.)} = T_L + 0.25\,(T_R' - T_R) = 15.1 + 0.25\,(3 - 2.75) = 15.2 \text{ HRS}$$

$$Q_P = \frac{640\ C_P A}{T_{L\,(adj.)}} = \frac{640(0.63)875}{15.2} = 23,210 \text{ CFS (657 CMS)}$$

$$T_B = 3 + \frac{T_{L\,(adj.)}}{8} = 3 + \frac{15.2}{8} = 4.9 \text{ DAYS} = 117.6 \text{ HRS}$$

$$W_{50} = 735\left(\frac{Q_P}{A}\right)^{-1.075} = 735\left(\frac{23,210}{875}\right)^{-1.075} = 21.7 \text{ HRS}$$

$$W_{75} = 434\left(\frac{Q_P}{A}\right)^{-1.075} = 434\left(\frac{23,210}{875}\right)^{-1.075} = 12.8 \text{ HRS}$$

Compared to the hydrograph widths at 50 and 75 percent of the peak flow, a time base of 117.6 hours is very long. To obtain a more realistic value, it is assumed that the time base is 4.5 times the time to peak, or

$$T_B = 4.5\,(T_{L\,(adj.)} + T_R/2) = 4.5\,(15.2 + 2.75/2) = 74.6 \text{ HRS}$$

These points are plotted in Figure 50 and a smooth hydrograph shape is fitted with the key dimensions. The volume under the hydrograph is then computed as shown in Table 34, with the discharge ordinates being scaled from the figure. The total volume computed is 1.128 inches, which is larger than the required 1 inch. The surplus volume, over 1 inch, must be deducted from the unit hydrograph in a reasonable and systematic way. The procedure described below is recommended for the following reasons:

1. The time to peak and peak discharge are preserved,

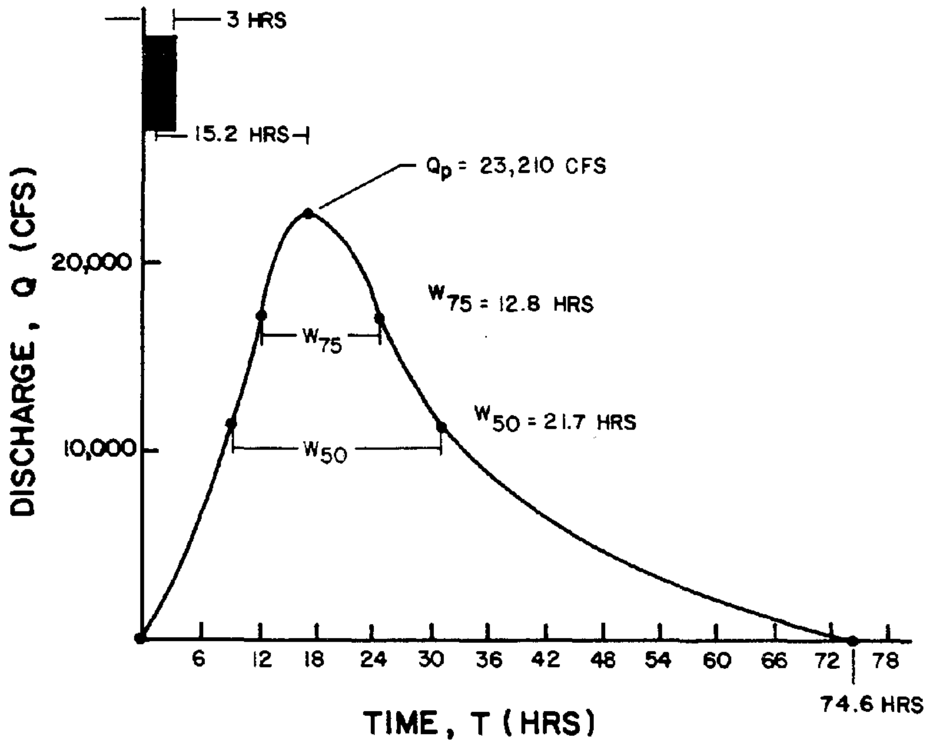



Figure 50. Snyder Unit Hydrograph for 3-Hour Duration

2. The bulk of the volume is deducted from the recession limb of the hydrograph, which is more uncertain than the rest of the hydrograph, and

3. The time base is affected, but is only approximated by the Equation (6-5).

Beginning at a convenient point near the $W_{50}$ point on the recession limb of the hydrograph, the discharge is decreased linearly according to the equation

$$Q' = Q\left[1 - a\left(\frac{T - T_i}{T_o - T_i}\right)\right] \qquad (6\text{-}8)$$

where Q' is the the adjusted discharge in CFS, Q is the the original discharge in CFS, $T_i$ is the time when the adjustment begins in hrs, T is the time associated with current discharge in hrs, $T_o$ is the time at the end of the hydrograph in hrs, and a is a constant determined by trial and error.

NOTE: Q' cannot be less than zero. If Q' is calculated to be less than zero using the equation above, it is set equal to zero.

Table 34. Direct Runoff Volume for Snyder Unit Hydrograph

| Time HRS | Δ Time HRS | Unit Hydrograph Discharge CFS | Average Unit Hydrograph Discharge CFS | Incremental Volume IN | Cumulative Volume IN |
|---|---|---|---|---|---|
| 0 | | 0 | | | |
| | 6 | | 3,165 | 0.034 | 0.034 |
| 6 | | 6,330 | | | |
| | 6 | | 11,415 | 0.121 | 0.155 |
| 12 | | 16,500 | | | |
| | 3 | | 19,365 | 0.103 | 0.258 |
| 15 | | 22,230 | | | |
| | 3 | | 22,615 | 0.120 | 0.378 |
| 18 | | 23,000 | | | |
| | 3 | | 22,165 | 0.118 | 0.496 |
| 21 | | 21,330 | | | |
| | 3 | | 20,180 | 0.107 | 0.603 |
| 24 | | 19,030 | | | |
| | 6 | | 15,880 | 0.169 | 0.772 |
| 30 | | 12,730 | | | |
| | 6 | | 10,880 | 0.116 | 0.888 |
| 36 | | 9,030 | | | |
| | 6 | | 7,830 | 0.083 | 0.971 |
| 42 | | 6,630 | | | |
| | 6 | | 5,880 | 0.062 | 1.033 |
| 48 | | 5,130 | | | |
| | 6 | | 4,230 | 0.045 | 1.078 |
| 54 | | 3,330 | | | |
| | 6 | | 2,695 | 0.029 | 1.107 |
| 60 | | 2,060 | | | |
| | 6 | | 1,480 | 0.016 | 1.123 |
| 66 | | 900 | | | |
| | 6 | | 515 | 0.005 | 1.128 |
| 72 | | 130 | | | |
| | 1.9 | | 65 | 0.000 | 1.128 |
| 74.6 | | 0 | | | |

The application of this procedure is best illustrated using the synthetic unit hydrograph from above. There is a need to deduct 0.128 inches from the volume of runoff. A point near the $W_{50}$ point on the recession limb of the hydrograph is chosen, in this case, the 30-hour point. Then Equation (6-8) is used to decrease the discharges subsequent to the 30-hour point, as follows

$$Q' = Q\left[1 - a\left[\frac{T-30}{74.6-30}\right]\right]$$

The constant "a" must be chosen by trial and error as demonstrated below: For a value of a = 1.0, determine the volume of the adjusted synthetic unit hydrograph as shown in Table 35.

Table 35. Direct Runoff Volume Adjustment for Snyder Unit Hydrograph

| Time HRS | Δ Time HRS | Unit Hydrograph Discharge CFS | Adjusted Unit Hydrograph Discharge CFS | Average Unit Hydrograph Discharge CFS | Incremental Volume IN | Cumulative Volume IN |
|---|---|---|---|---|---|---|
| 0 | | 0 | 0 | | | |
| | 6 | | | 3,165 | 0.034 | 0.034 |
| 6 | | 6,330 | 6,330 | | | |
| | 6 | | | 11,415 | 0.121 | 0.155 |
| 12 | | 16,500 | 16,500 | | | |
| | 3 | | | 19,365 | 0.103 | 0.258 |
| 15 | | 22,230 | 22,230 | | | |
| | 3 | | | 22,615 | 0.120 | 0.378 |
| 18 | | 23,000 | 23,000 | | | |
| | 3 | | | 22,165 | 0.118 | 0.496 |
| 21 | | 21,300 | 21,300 | | | |
| | 3 | | | 20,180 | 0.107 | 0.603 |
| 24 | | 19,030 | 19,030 | | | |
| | 6 | | | 15,880 | 0.169 | 0.772 |
| 30 | | 12,730 | 12,730 | | | |
| | 6 | | | 10,263 | 0.109 | 0.881 |
| 36 | | 9,030 | 7,796 | | | |
| | 6 | | | 6,307 | 0.067 | 0.948 |
| 42 | | 6,630 | 4,818 | | | |
| | 6 | | | 3,922 | 0.042 | 0.990 |
| 48 | | 5,130 | 3,027 | | | |
| | 6 | | | 2,263 | 0.024 | 1.014 |
| 54 | | 3,330 | 1,510 | | | |
| | 6 | | | 1,081 | 0.011 | 1.025 |
| 60 | | 2,060 | 652 | | | |
| | 6 | | | 407 | 0.005 | 1.030 |
| 66 | | 900 | 162 | | | |
| | 6 | | | 84 | 0.001 | 1.031 |
| 72 | | 130 | 6 | | | |
| | 1.9 | | | 3 | 0.000 | 1.031 |
| 74.6 | | 0 | 0 | | | |

The volume is still too high. Several other values of "a" can be tried until the volume under the unit hydrograph equals 1 inch. These trials are tabulated below

| a | Total Volume |
|---|---|
| 1.0 | 1.031 |
| 1.1 | 1.021 |
| 1.3 | 1.002 |

A value of $a = 1.3$ produces a runoff volume within less than one percent of the required 1 inch. The final synthetic unit hydrograph is shown in Figure 51 below, together with the original synthetic unit hydrograph to illustrate the volume adjustment.

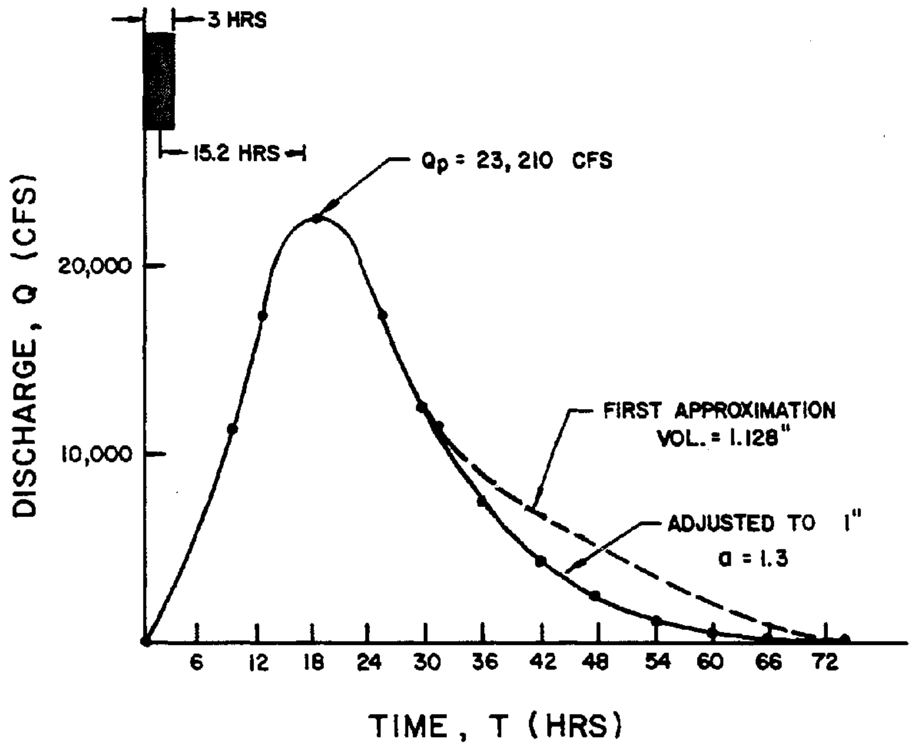



Figure 51. Adjusted 3-Hour Snyder Unit Hydrograph

The final unit hydrograph is a 3-hour unit hydrograph for the 875 square mile watershed. It can be used in the same manner as a unit hydrograph derived from gage records.

### 6.2.2 SCS Synthetic Unit Hydrograph

The Soil Conservation Services, SCS Handbook, 1972, has developed a synthetic unit hydrograph procedure which has been widely used in their conservation and flood control work. The unit hydrograph used by the SCS is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions. This method is easy to apply. The only parameters which need be determined are the peak discharge and the time to peak. With these two parameters, a standard unit hydrograph is constructed which can then be used in the same manner as the unit hydrographs previously presented.

A step-by-step procedure for applying the SCS unit hydrograph method is given below:

1. <u>Determine the time to peak, $T_p$</u>

The time to peak is defined as the time from the beginning of rainfall to the peak discharge. This is determined using the equation below

$$T_p = \frac{D}{2} + T_L \qquad (6\text{-}9)$$

where $T_p$ is the time to peak in hrs, D is the duration of excess rainfall in hrs, and $T_L$ is the lag time or the time from the centroid of excess rainfall to the peak discharge in hrs.

The SCS recommends that D be taken as 0.133 of the time of concentration of the watershed, $T_c$. In other words

$$D = 0.133\ T_c \qquad (6\text{-}10)$$

This recommendation is based upon the characteristics of the curvilinear unit hydrograph developed by the SCS and should not be disregarded.

The SCS also estimates that the lag time, $T_L$, is related to the time of concentration of the watershed by the empirical equation

$$T_L = 0.6\ T_c \qquad (6\text{-}11)$$

Therefore, the time to peak, $T_p$, is given as

$$T_p = 0.67\ T_c \qquad (6\text{-}12)$$

The time of concentration for the watershed is defined as the time it takes for runoff to travel from the most hydraulically remote point in the watershed to the point of interest, usually the outlet of the watershed.

The SCS gives three methods for determining $T_c$ for a watershed as summarized below.

6.2.2.1 <u>Stream Hydraulic Method</u>

Based upon field survey data, topographic maps and any other information which is available, the designer determines the longest watercourse within the watershed of interest. This watercourse is then subdivided into relatively uniform reaches. The travel time of each reach is based upon the average velocity of the bankfull discharge. Manning's Equation is used to compute the velocity. The sum of the travel times for all the reaches, up to the watershed divide, is taken to be the time of concentration of the watershed. For the usual case, where a definable channel does not extend to the watershed divide, the last increment of travel time can be estimated using either of the procedures summarized below, whichever is more applicable.

6.2.2.2 Upland Method

The types of flow covered by the upland method are: overland, through grassed waterways, over paved areas, through small upland gullies, and along terrace channels. The velocity for upland flow is determined from Figure 52.

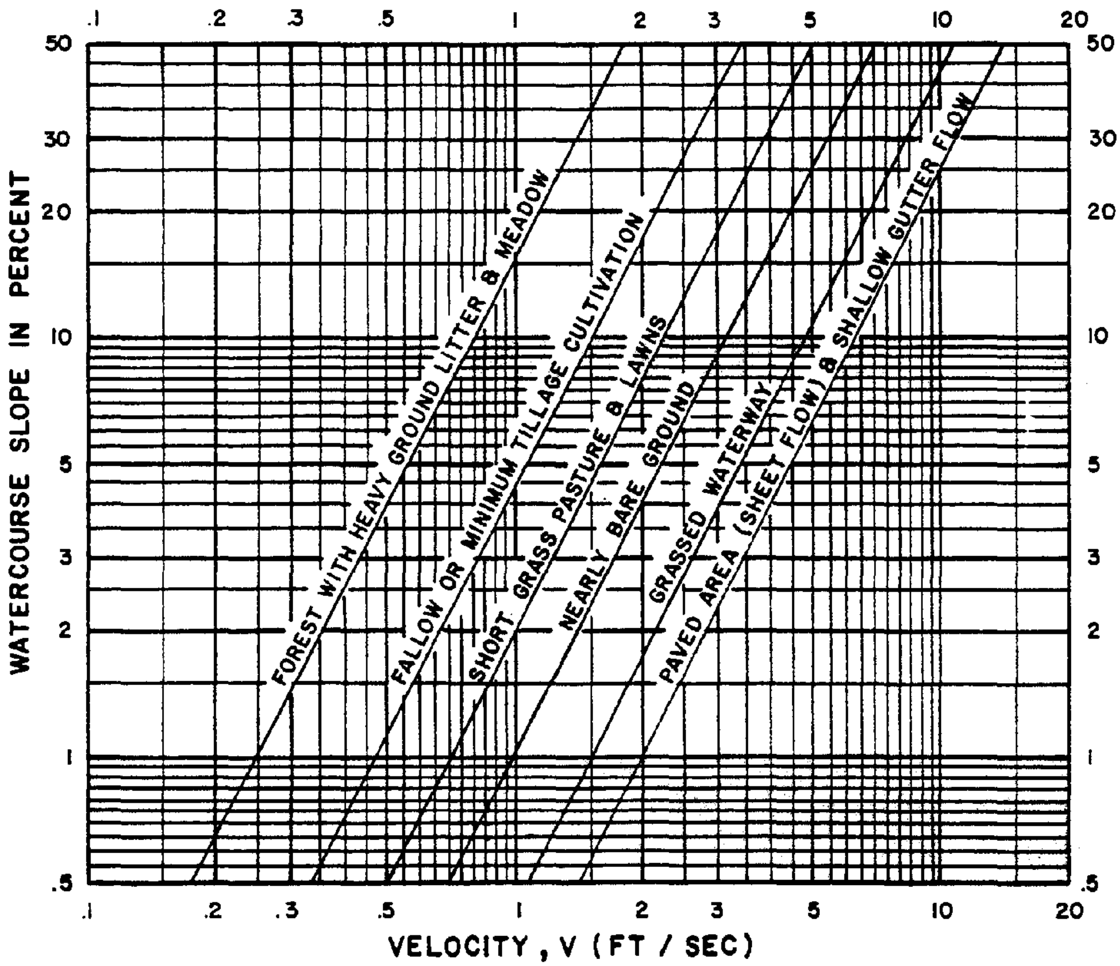



Figure 52. Velocities for Upland Method of Estimating $T_C$

The travel time is then simply computed using the equation below

$$T_t = \frac{L}{3600\,V} \tag{6-13}$$

where $T_t$ is the travel time in hrs, L is the hydraulic length in ft, and V is the velocity in feet per second.

The upland method is applicable only to small watersheds, or subwatersheds (2000 acres or less) and to the types of flow listed above.

6.2.2.3 Curve Number Method

This method is based upon data from the SCS (ARS) research watersheds, and is summarized in the equation below

$$T_c = \frac{L^{0.8}\,(S+1)^{0.7}}{1140\,Y^{0.5}} \tag{6-14}$$

where $T_c$ is the time of concentration in hrs, L is the length to the watershed divide in feet, S is the potential maximum retention in inches which is equal to $(\frac{1000}{CN} - 10)$ where CN is the SCS curve number for the watershed, and Y is the average watershed slope in percent.

The curve number, CN, is determined by an evaluation of soil type, antecedent moisture conditions and land use. To determine CN, the soil is first classified by the SCS into a hydrologic soil group in accordance with Table 36.

The SCS Handbook, 1972, also includes a list giving the Hydrologic Soil group for over 4000 soil types in the United States and Puerto Rico.

The hydrologic condition of the soil is determined primarily by soil management practices. In the case of farm and pasture land, the condition is defined as:

Poor - Heavily grazed, no mulch or less than one-half the area covered with vegetation.

Fair - Moderately grazed, one-half to three-fourths of the area covered by vegetation.

Good - Lightly grazed, more than three-fourths of the area covered by vegetation.

Table 36. Hydrologic Soil Group Descriptions

| |
|---|
| A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission. |
| B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission. |
| C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly 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. |
| D. (High runoff potential). Soils having very low infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water content and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission. |

from SCS, 1972.

Antecedent Moisture Conditions (AMC) are also grouped into three categories as follows:

AMC I - Low moisture, soil is dry.

AMC II - Average moisture conditions. Condition normally used for annual flood estimates.

AMC III - High moisture, heavy rainfall over preceding few days.

With the hydrologic soil group, soil condition and antecedent moisture conditions of AMC II, the value of CN can be obtained from Table 37.

Table 38 can be used to obtain curve numbers for other antecedent moisture conditions (I and III).

The curve number method is also limited to small watersheds, or subwatersheds (less than 2000 acres) but does apply to a broad range of conditions, ranging from heavily forested to smooth land surfaces and large paved areas. It is emphasized that the above descriptions of these procedures are merely summaries. The reader is referred to the SCS National Engineering Handbook, Section 4, Hydrology, for a more detailed description of the procedures.

Table 37. Runoff Curve Numbers for Hydrologic Soil-Cover Complexes (Antecedent moisture condition II)

| Land Use | Treatment or Practice | Hydrologic Condition | Hydrologic Soil Group A | B | C | D |
|---|---|---|---|---|---|---|
| Fallow | Straight Row | ---- | 77 | 86 | 91 | 94 |
| Row Crops | " | Poor | 72 | 81 | 88 | 91 |
| | " | Good | 67 | 78 | 85 | 89 |
| | Contoured | Poor | 70 | 79 | 84 | 88 |
| | " | Good | 65 | 75 | 82 | 86 |
| | "and Terraced | Poor | 66 | 74 | 80 | 82 |
| | " " " | Good | 62 | 71 | 78 | 81 |
| Small Grain | Straight Row | Poor | 65 | 76 | 84 | 88 |
| | | Good | 63 | 75 | 83 | 87 |
| | Contoured | Poor | 63 | 74 | 82 | 85 |
| | | Good | 61 | 73 | 81 | 84 |
| | "and Terraced | Poor | 61 | 72 | 79 | 82 |
| | | Good | 59 | 70 | 78 | 81 |
| Close-seeded legumes 1/ or rotation meadow | Straight row | Poor | 66 | 77 | 85 | 89 |
| | " " | Good | 58 | 72 | 81 | 85 |
| | Contoured | Poor | 64 | 75 | 83 | 85 |
| | " | Good | 55 | 69 | 78 | 83 |
| | "and terraced | Poor | 63 | 73 | 80 | 83 |
| | "and terraced | Good | 51 | 67 | 76 | 80 |
| Pasture or range | | Poor | 68 | 79 | 86 | 89 |
| | | Fair | 49 | 69 | 79 | 84 |
| | | Good | 39 | 61 | 74 | 80 |
| | Contoured | Poor | 47 | 67 | 81 | 88 |
| | " | Fair | 25 | 59 | 75 | 83 |
| | " | Good | 6 | 35 | 70 | 79 |
| Meadow | | Good | 30 | 58 | 71 | 78 |
| Woods | | Poor | 45 | 66 | 77 | 83 |
| | | Fair | 36 | 60 | 73 | 79 |
| | | Good | 25 | 55 | 70 | 77 |
| Farmsteads | | ---- | 59 | 74 | 82 | 86 |
| Roads (dirt) 2/ | | ---- | 72 | 82 | 87 | 89 |
| (hard surface) 2/ | | ---- | 74 | 84 | 90 | 92 |

1/ Close-drilled or broadcast.

2/ Including right-of-way.

from SCS, 1972

Table 38. Values of CN for Other Antecedent Moisture Conditions

| CN for AMC II | CN for AMC I | CN for AMC III |
|---|---|---|
| 100 | 100 | 100 |
| 95 | 87 | 99 |
| 90 | 78 | 98 |
| 85 | 70 | 97 |
| 80 | 63 | 94 |
| 75 | 57 | 91 |
| 65 | 45 | 83 |
| 60 | 40 | 79 |
| 55 | 35 | 75 |
| 50 | 31 | 70 |
| 45 | 27 | 65 |
| 40 | 23 | 60 |
| 35 | 19 | 55 |
| 30 | 15 | 50 |
| 25 | 12 | 45 |
| 20 | 9 | 39 |
| 15 | 7 | 33 |
| 10 | 4 | 26 |
| 5 | 2 | 17 |
| 0 | 0 | 0 |

from SCS, 1972

Once the time to peak has been determined, the next step of the process is to determine the peak discharge.

2. Determine Peak Discharge

The peak discharge of the synthetic unit hydrograph is determined using the equation below:

$$q_p = \frac{K_p A}{T_p} \qquad (6\text{-}15)$$

where $q_p$ is the peak discharge in CFS, A is the drainage area in sq mi, $T_p$ is the time to peak in hrs, and $K_p$ is an empirical constant which varies from 300 in very flat swampy areas to 600 in steep terrains. An average value of 484 is used unless otherwise indicated.

Once the two parameters, $T_p$ and $q_p$ have been computed, the synthetic unit hydrograph can be determined using the dimensionless unit hydrograph coordinates given in Table 39. This same information is shown graphically in Figure 53.

This dimensionless unit hydrograph is typically used for $K_p$ values equal to 484. If $K_p$ differs significantly from 484, then the shape of the dimensionless unit hydrograph will be different. A local SCS office should be contacted for guidance in such cases. These offices are listed in Appendix C.

Table 39. Ratios for Dimensionless Unit Hydrograph and Mass Curve, SCS Synthetic Hydrograph

| Time Ratios ($t/T_p$) | Discharge Ratios ($q/q_p$) | Mass Curve Ratios ($Q_a/Q$) |
|---|---|---|
| .0 | .000 | .000 |
| .1 | .030 | .001 |
| .2 | .100 | .006 |
| .3 | .190 | .012 |
| .4 | .310 | .035 |
| .5 | .470 | .065 |
| .6 | .660 | .107 |
| .7 | .820 | .163 |
| .8 | .930 | .228 |
| .9 | .990 | .300 |
| 1.0 | 1.000 | .375 |
| 1.1 | .990 | .450 |
| 1.2 | .930 | .522 |
| 1.3 | .860 | .589 |
| 1.4 | .780 | .650 |
| 1.5 | .680 | .700 |
| 1.6 | .560 | .751 |
| 1.7 | .460 | .790 |
| 1.8 | .390 | .822 |
| 1.9 | .330 | .849 |
| 2.0 | .280 | .871 |
| 2.2 | .207 | .908 |
| 2.4 | .147 | .934 |
| 2.6 | .107 | .953 |
| 2.8 | .077 | .967 |
| 3.0 | .055 | .977 |
| 3.2 | .040 | .984 |
| 3.4 | .029 | .989 |
| 3.6 | .021 | .993 |
| 3.8 | .015 | .995 |
| 4.0 | .011 | .997 |
| 4.5 | .005 | .999 |
| 5.0 | .000 | 1.000 |

from SCS, 1972

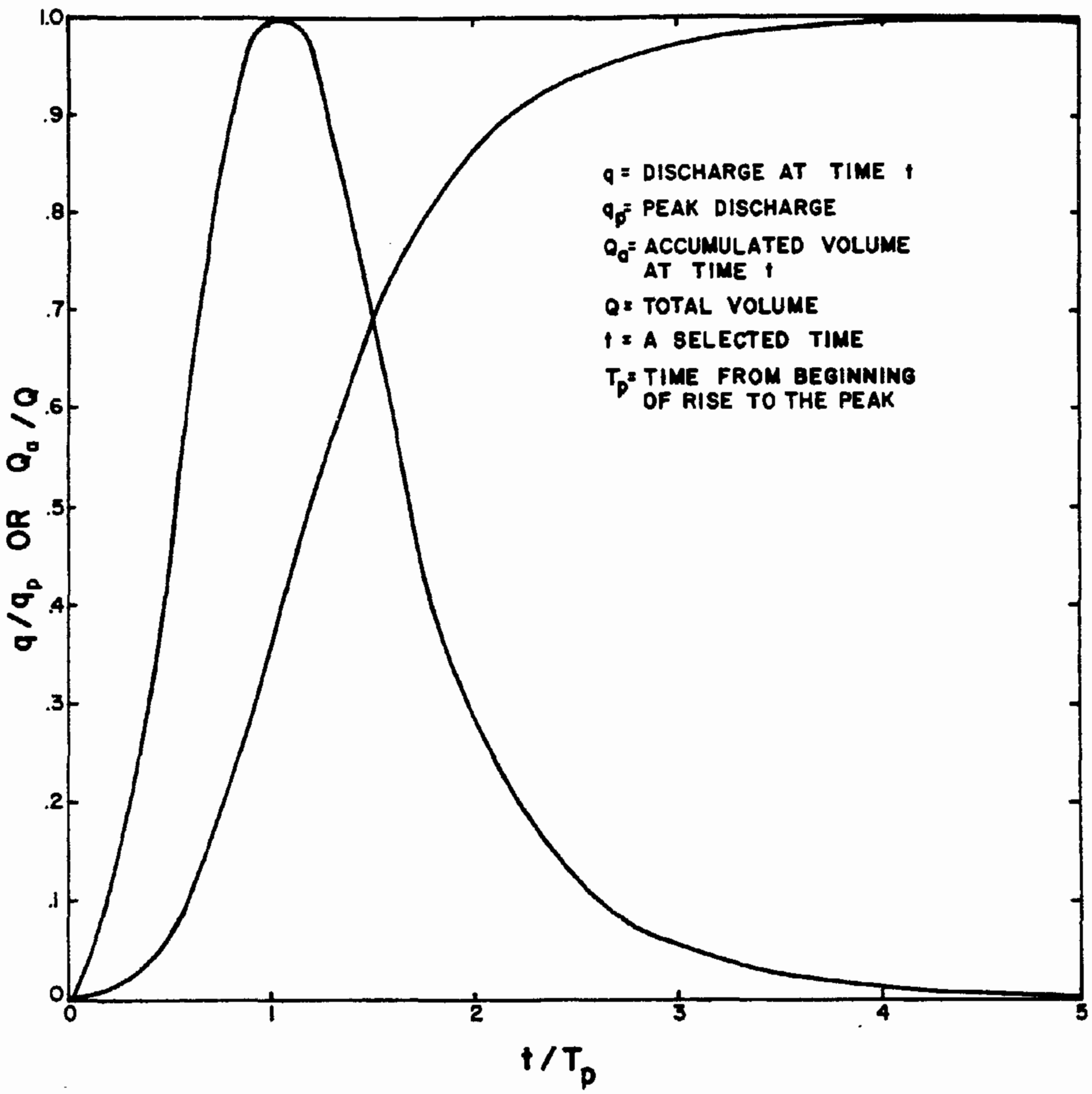



Figure 53. Dimensionless Unit Hydrograph and Mass Curve for SCS Synthetic Hydrograph

### 6.2.3 SCS Synthetic Triangular Hydrograph

A characteristic of the dimensionless unit hydrograph shown in Figure 53 is that it has 37.5 percent of the runoff volume (1-inch) under the rising limb. An equivalent triangular unit hydrograph can be constructed as shown in Figure 54 such that it also has 37.5 percent of the volume under the rising side of the triangle.

Using the triangle geometry, the time base for the unit hydrograph can be calculated as

$$T_b = \frac{1.000}{.375} = 2.67\ T_p \tag{6-16}$$

and

$$T_r = T_b - T_p = 1.67\ T_p \tag{6-17}$$

where $T_b$, $T_r$ and $T_p$ are defined as shown in Figure 54. The volume of runoff can also be computed from Figure 54 as

$$Q = \frac{q_p T_p}{2} + \frac{q_p T_r}{2} = \frac{q_p}{2}(T_p + T_r) \tag{6-18}$$

and the peak flow is

$$q_p = \frac{2Q}{(T_p + T_r)} = \frac{2Q}{T_p(1 + \frac{T_r}{T_p})} = \frac{KQ}{T_p} \tag{6-19}$$

where Q is the volume (equal to one inch for the unit hydrograph), $q_p$ is the peak flow and the coefficient, $K = 2/(1 + \frac{T_r}{T_p})$. Converting the units in Equation (6-19) to T (hours), $q_p$ (cfs) and A (sq mi) gives

$$q_p = 645.33\,K\,\frac{(AQ)}{T_p} \tag{6-20}$$

The factor 645.33 is the rate necessary to discharge one-inch of runoff from 1 square mile in 1 hour. Using $T_r = 1.67\ T_p$ gives K = 0.75, and Equation (6-20) reduces to

$$q_p = \frac{484\,AQ}{T_p} \tag{6-21}$$

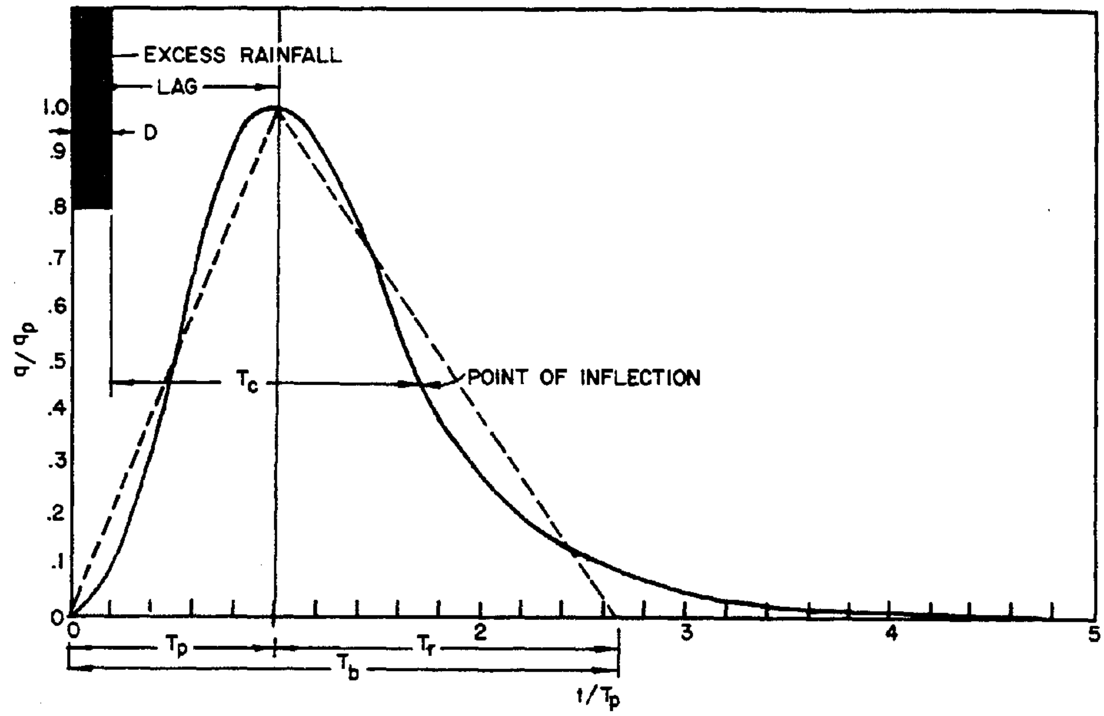



Figure 54. Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hyrograph

Equation (6-21) is identical to Equation (6-15) with an average $K_p$ of 484 given for the SCS dimensionless unit hydrograph, Figure 53. Other characteristics necessary to complete the triangular unit hydrograph, namely, time to peak, $T_p$, duration of excess rainfall, D, lag time, $T_L$, and time of concentration, $T_c$ are computed by the methods described in Section 6.2.2.

The triangular unit hydrograph is simple to work with because of the linearity of the rising and falling limbs and requires less computational effort than the SCS dimensionless unit hydrograph. The primary difference between the two methods is in the length of the time base. The triangular hydrograph has a time base of 2.67 units of time compared to the dimensionless unit hydrograph which has a time base of 5.0. This difference, however, is relatively unimportant. As seen in Figure 54, this difference occurs at the recession limb of the hydrograph when the flows are small and the major part of the surface runoff has already occurred. Because of the shorter time base, the use of the triangular unit hydrograph in evaluating complex storms, will tend to give slightly lower peak flood flows compared to the SCS dimensionless unit hydrograph but gives excellent agreement on the time to major and secondary peaks.

To illustrate the development of a unit hydrograph by the SCS dimensionless and triangular methods, consider the same data used for the Snyder unit hydrograph method, Section 6.2.1.

The drainage area is 875 square miles and the longest hydraulic length is 83 miles. In addition to this information, it is also known that the upper 2 miles of this length is overland flow (forest with heavy ground cover) at a slope of 4 percent. The remaining 81 miles is a clean dredged channel with a Manning roughness coefficient of 0.022 and an average slope of 1 foot per mile. The channel is wide and the hydraulic radius may be taken as the average bank full depth of 15 feet.

Using this information, a unit hydrograph can be developed as follows.

1. Calculate the time of concentration ($T_c$) for the watershed. This calculation is very important in hydrograph development because the time base and peak flow are affected by this quantity.

   a. Using the Upland Method mentioned previously and Figure 52, $T_c$ for the overland flow can be estimated. For forest with heavy cover @ 4 percent slope -- V = 0.5 FPS (0.15 MPS).

   $$Tc = \frac{L}{V} = \frac{2\text{ mi}}{.5^{\text{ft}}/\text{sec}} \times \frac{5280\text{ft}}{\text{mi}} \times \frac{\text{HR}}{3600\text{sec}} = 5.9 \text{ HRS}$$

   b. The Manning equation is used to analyze the remainder of the channel reach as follows

   $$V = \frac{1.49}{n}(R)^{0.667}(S)^{0.5} = \frac{1.49}{0.022}(15)^{0.667}\left(\frac{1}{5280}\right)^{0.5} = 5.67 \text{ FPS } (1.73 \text{ MPS})$$

   The time of concentration for this reach is

   $$Tc = \frac{L}{V} = \frac{81\text{ mi}}{5.66^{\text{ft}}/\text{sec}} \times \frac{5280\text{ ft}}{\text{mi}} \times \frac{\text{HR}}{3600\text{ sec}} = 21.0 \text{ HRS}$$

   c. The total time of concentration for the basin is

   $$\Sigma\, Tc = 5.9 + 21.0 = 26.9 \text{ HRS}$$

2. Calculate $T_p$ and $q_p$

   $$T_p = .67\, Tc = .67(26.9) = 18.0 \text{ HRS}$$

   $$q_p = \frac{484A}{T_p} = \frac{(484)(875)}{18.0} = 23{,}528 \text{ CFS (666 CMS)}$$

3. Calculate $T_b$ and $T_r$ for Triangular and Dimensionless hydrograph

   a. Triangular Hydrograph

$$T_b = 2.67\ T_p = 2.67(18.0) = 48.1 \text{ HRS}$$
$$T_r = T_b - T_p = 48.1 - 18.0 = 30.1 \text{ HRS}$$

   b. Dimensionless Hydrograph

$$T_b = 5\ T_p = 5(18.0) = 90.0 \text{ HRS}$$
$$T_r = T_b - T_p = 90.0 - 18.0 = 72.0 \text{ HRS}$$

4. Calculate the other parameters of the unit hydrograph which are common to both the triangular and dimensionless unit graphs.

$$D = .133\ T_c = .133(26.9) = 3.6 \text{ HRS}$$
$$T_L = .6 T_c = .6(26.9) = 16.1 \text{ HRS}$$

5. Plot unit hydrographs as shown in Figure 55.

   a. The Triangular hydrograph is plotted using $T_p$, $q_p$, and $T_r$.

   b. The hydrograph determined from the dimensionless ratios is plotted using $T_p$, $q_p$ and Table 40.

### 6.2.4 Transposition of Unit Hydrographs

Another method that can be used to develop a unit hydrograph at an ungaged site is to transpose unit hydrographs from other hydrologically homogeneous watersheds. The four basic factors needed to identify a hydrograph are the peak flow, time to peak, duration of flow or time base and the volume of runoff.

In transposing hydrographs, time to peak is defined by the lag or the time from the midpoint of the excess rainfall duration to the time of the peak flow. Lag can be defined by the equation

$$LAG = C\left(\frac{LL_{ca}}{Y^{1/2}}\right)^K \qquad (6\text{-}22)$$

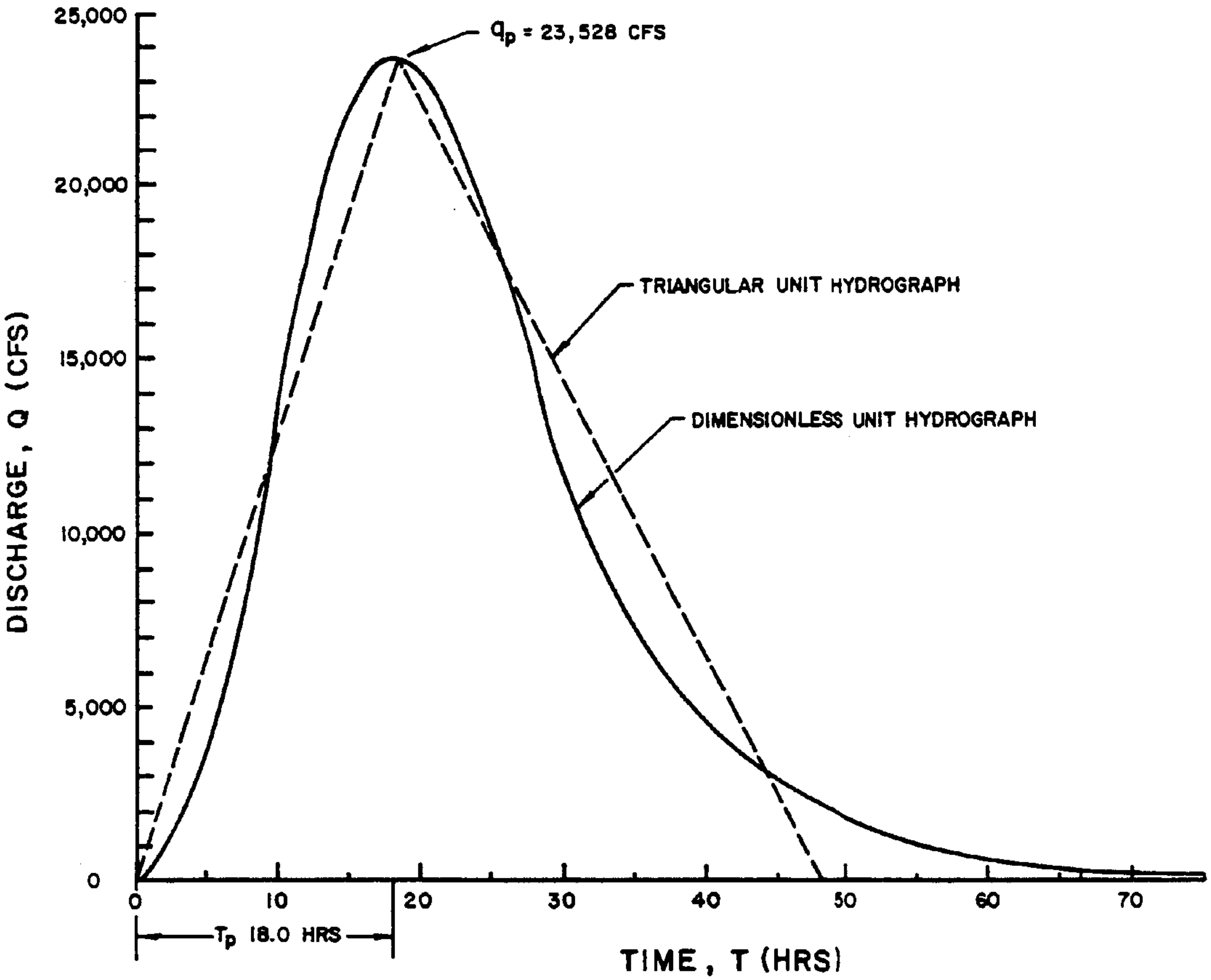



Figure 55. SCS Unit Hydrographs by Dimensionless Ratio and Triangular Methods

where L is the length of the longest watercourse, mi, $L_{ca}$ is the length along the longest watercourse from the outlet to a point opposite the centroid of the basin, mi, Y is the slope of the longest watercourse in percent and C and K are coefficients to be determined from the hydrologically homogeneous areas. The coefficients in Equation (6-22) and the lag for the ungaged site can be determined from a full logarithmic plot of lag vs (L $L_{ca}/Y^{1/2}$). The peak flow of the unit hydrograph can be determined in the same manner by logarithmically correlating peak flow with drainage area.

The duration of flow is best determined by converting each unit hydrograph into a dimensionless form by dividing the flows and times by the respective peak flow and lag for each basin. These dimensionless hydrographs can then be plotted to obtain an average value for the time base. The shape of the unit graph is then estimated from the transposed hydrographs and the volume

Table 40. Calculations of SCS Synthetic Unit Hydrograph

| Time Ratios ($t/T_p$) | Time (hrs) | Discharge Ratios ($q/q_p$) | Q (CFS) | Mass Curve Ratios ($Q_a/Q$) |
|---|---|---|---|---|
| .0 | 0.0 | .000 | 0 | .000 |
| .1 | 1.8 | .030 | 706 | .001 |
| .2 | 3.6 | .100 | 2,353 | .006 |
| .3 | 5.4 | .190 | 4,470 | .012 |
| .4 | 7.2 | .310 | 7,294 | .035 |
| .5 | 9.0 | .470 | 11,058 | .065 |
| .6 | 10.8 | .660 | 15,528 | .107 |
| .7 | 12.6 | .820 | 19,293 | .163 |
| .8 | 14.4 | .930 | 21,881 | .228 |
| .9 | 16.2 | .990 | 23,293 | .300 |
| 1.0 | 18.0 | 1.000 | 23,528 | .375 |
| 1.1 | 19.8 | .990 | 23,293 | .450 |
| 1.2 | 21.6 | .930 | 21,881 | .522 |
| 1.3 | 23.4 | .860 | 20,234 | .589 |
| 1.4 | 25.2 | .780 | 18,352 | .650 |
| 1.5 | 27.0 | .680 | 15,999 | .700 |
| 1.6 | 28.8 | .560 | 13,176 | .751 |
| 1.7 | 30.6 | .460 | 10,823 | .790 |
| 1.8 | 32.4 | .390 | 9,176 | .822 |
| 1.9 | 34.2 | .330 | 7,764 | .849 |
| 2.0 | 36.0 | .280 | 6,588 | .871 |
| 2.2 | 39.6 | .207 | 4,870 | .908 |
| 2.4 | 43.2 | .147 | 3,459 | .934 |
| 2.6 | 46.8 | .107 | 2,517 | .953 |
| 2.8 | 50.4 | .077 | 1,812 | .967 |
| 3.0 | 54.0 | .055 | 1,294 | .977 |
| 3.2 | 57.6 | .040 | 941 | .984 |
| 3.4 | 61.2 | .029 | 682 | .989 |
| 3.6 | 64.8 | .021 | 494 | .993 |
| 3.8 | 68.4 | .015 | 353 | .995 |
| 4.0 | 72.0 | .011 | 259 | .997 |
| 4.5 | 81.0 | .005 | 118 | .999 |
| 5.0 | 90.0 | .000 | 0 | 1.000 |

checked to ensure it represents 1-inch of runoff from the basin of interest. If not, the shape is adjusted until the volume is reasonably close to 1-inch. This transposition procedure is illustrated in the design hydrograph example given in Sec. 6.4.2.

## 6.3 SCS Peak Flow Estimates

In Section 5.0, it was noted that the SCS presents curves from which peak flows could be estimated for particular types of rainfall distributions. In the application of the Soil-Cover-Complex method to develop unit hydrographs and to estimate surface runoff from agricultural and urban watersheds, the Soil Conservation Service, 1972, 1975 presents a graphical method for determining peak discharges. The soil-cover-complex and its determination was discussed in detail in Section 6.2.2.3.

The soil-cover-complex is a combination of a hydrologic soil group which characterizes the soil conditions and a land use and treatment class which is a descriptor of ground cover. The effect of the soil-cover-complex on the excess rainfall or the amount of precipitation that runs off is represented by a Runoff Curve Number referred to as the CN.

In order to determine the direct runoff (excess rainfall) from a given depth of precipitation and the curve number, the SCS, 1972 develops the relation

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \tag{6-23}$$

where Q is the direct runoff in inches, P is the depth of precipitation, $I_a$ is the initial abstraction in inches and S is the storage in the watershed in inches. In Equation (6-23), S and $I_a$ are given by the relations

$$S = \frac{1000}{CN} - 10 \tag{6-24}$$

and

$$I_a = 0.2S \tag{6-25}$$

If Equation (6-25) is substituted into Equation (6-23) the following relation results

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{6-26}$$

The following Table 41, taken from SCS, 1972, is computed from Equation (6-26) and gives the actual depth of runoff (storm rainfall less initial abstractions) in inches for selected values of CN and rainfall amounts. This same data is often presented in graphical form as shown in Figures 56a and 56b.

Table 41. Runoff Depth, Q, in Inches for Selected CN's and Rainfall Amounts

| Rainfall, P, Inches | Curve Number (CN)[1] | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 98 |
| 1.0 | 0.00 | 0.00 | 0.00 | 0.03 | 0.08 | 0.17 | 0.32 | .56 | .79 |
| 1.2 | 0.00 | 0.00 | 0.03 | 0.07 | 0.15 | 0.28 | 0.46 | .74 | .99 |
| 1.4 | 0.00 | 0.02 | 0.06 | 0.13 | 0.24 | 0.39 | 0.61 | .92 | 1.18 |
| 1.6 | 0.01 | 0.05 | 0.11 | 0.20 | 0.34 | 0.52 | 0.76 | 1.11 | 1.38 |
| 1.8 | 0.03 | 0.09 | 1.17 | 0.29 | 0.44 | 0.65 | 0.93 | 1.29 | 1.58 |
| 2.0 | 0.06 | 0.14 | 0.24 | 0.38 | 0.56 | 0.80 | 1.09 | 1.48 | 1.77 |
| 2.5 | 0.17 | 0.30 | 0.46 | 0.65 | 0.89 | 1.18 | 1.53 | 1.96 | 2.27 |
| 3.0 | 0.33 | 0.51 | 0.72 | 0.96 | 1.25 | 1.59 | 1.98 | 2.45 | 2.78 |
| 4.0 | 0.76 | 1.03 | 1.33 | 1.67 | 2.04 | 2.46 | 2.92 | 3.43 | 3.77 |
| 5.0 | 1.30 | 1.65 | 2.04 | 2.45 | 2.89 | 3.37 | 3.88 | 4.42 | 4.76 |
| 6.0 | 1.92 | 2.35 | 2.80 | 3.28 | 3.78 | 4.31 | 4.85 | 5.41 | 5.76 |
| 7.0 | 2.60 | 3.10 | 3.62 | 4.15 | 4.69 | 5.26 | 5.82 | 6.41 | 6.76 |
| 8.0 | 3.33 | 3.90 | 4.47 | 5.04 | 5.62 | 6.22 | 6.81 | 7.40 | 7.76 |
| 9.0 | 4.10 | 4.72 | 5.34 | 5.95 | 6.57 | 7.19 | 7.79 | 8.40 | 8.76 |
| 10.0 | 4.90 | 5.57 | 6.23 | 6.88 | 7.52 | 8.16 | 8.78 | 9.40 | 9.76 |
| 11.0 | 5.72 | 6.44 | 7.13 | 7.82 | 8.48 | 9.14 | 9.77 | 10.39 | 10.76 |
| 12.0 | 6.56 | 7.32 | 8.05 | 8.76 | 9.45 | 10.12 | 10.76 | 11.39 | 11.76 |

[1] To obtain runoff depths for CN's and other rainfall amounts not shown in this table, use an arithmetic interpolation.

If the watershed has uniform characteristics (cover, soils, land use, etc.) and can be represented by a single Curve Number, CN, the peak discharge can be estimated from Figure 57 which gives the peak discharge in CFS/sq mi/in of rainfall (actual). This graphical procedure approximates some of the methods used to develop hydrographs by the SCS Technical Release 20, 1965. The application of Figure 57 is limited to the peak runoff from a 24-hour duration storm of a Type II distribution, SCS, 1973. The Type II storm is characteristic of continental or summer thunderstorms. The distribution is arranged with the greatest 30-minute rainfall at the midpoint of the 24-hour duration. The second largest 30-minute rainfall is placed in the next 30-minute increment and the third largest in the preceding 30-minute increment. This arrangement is continued until the two smallest 30-minute rainfalls fall at the beginning and end of the 24-hour duration.

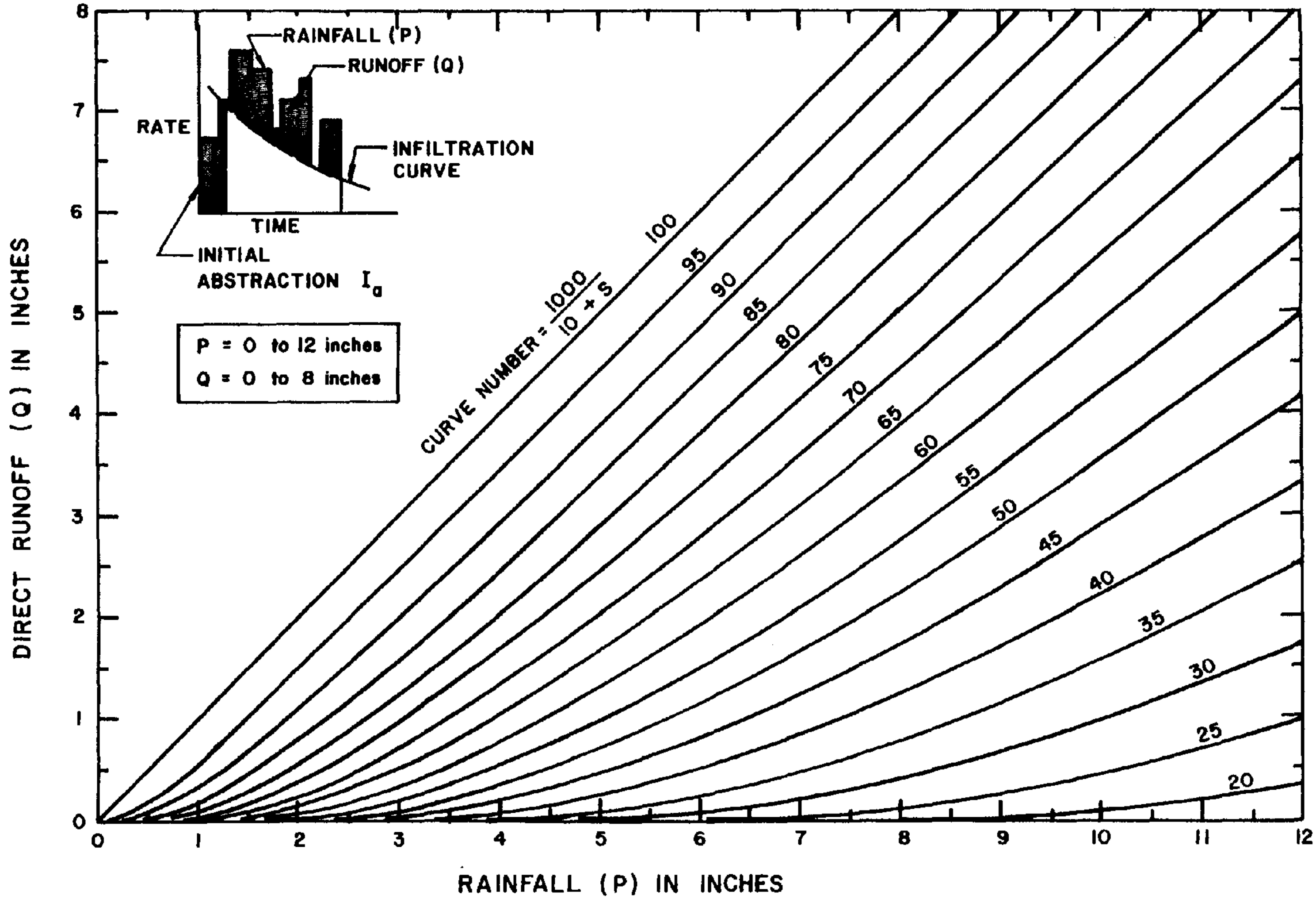



Figure 56a. SCS Relation Between Direct Runoff, Curve Number and Precipitation

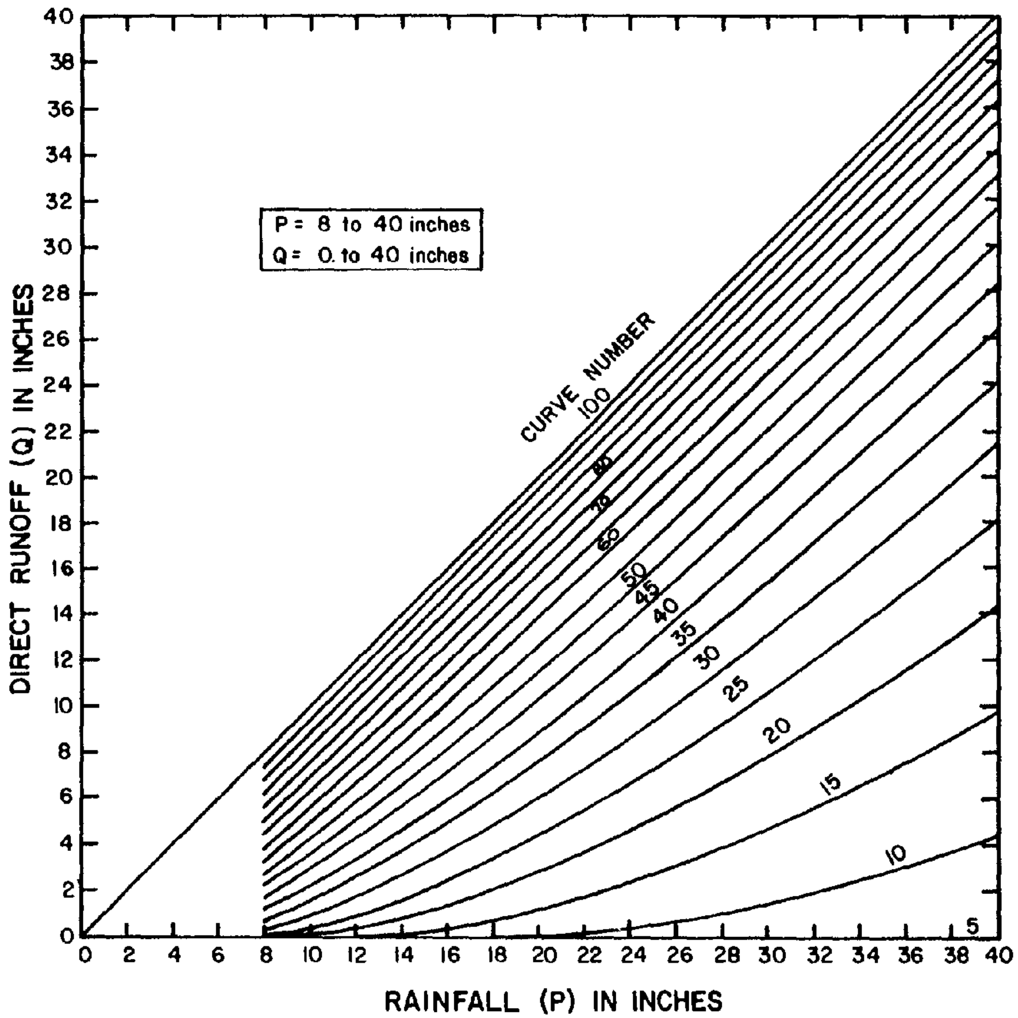



Figure 56b. SCS Relation Between Direct Runoff, Curve Number and Precipitation

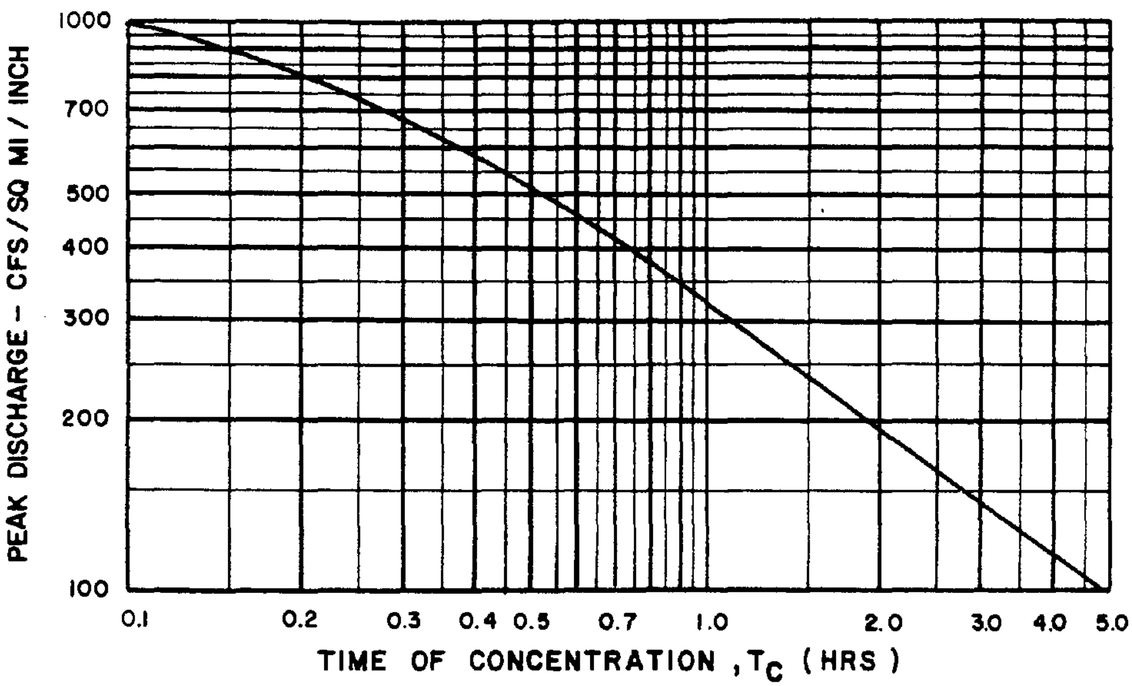



Figure 57. Peak Discharge as a Function of Time of Concentration

Figure 57 is also limited to watersheds where no routing of the hydrograph is required and where the travel time can be considered equal to zero.

As an example consider the following watershed:

Drainage Area = 1050 acres

Curve Number = 75

Time of Concentration = 1.1 hours

24-hour, 100-year Type II rainfall = 6.0 inches

From Table 41 for CN = 75 and rainfall = 6.0 inches,
the runoff depth = 3.28 inches

From Figure 57 for $T_c$ = 1.1 hours,
the peak discharge = 300 CFS/sq mi/inch

The 100-year peak flow is

$$Q = \frac{300 \text{ CFS}}{\text{SQ MI-IN}} \left( \frac{1050 \text{ ACRES}}{640 \text{ ACRES / SQ MI}} \right) (3.28 \text{ IN}) = 1614 \text{ CFS } (45.7 \text{ CMS})$$

For small watersheds with drainage areas less than 2000 acres, the SCS, 1975, also gives graphs for estimating peak discharge from a 24-hour duration Type II storm. These graphs, Figures 58, 59, and 60, relate the peak discharge in CFS/inch to drainage area in acres for various Curve Numbers and for flat, moderate and steep slopes. The curves are used in conjunction with Table 41 or Figures 56a and 56b for the depth of runoff and apply to agricultural watersheds or watersheds in their natural condition.

The methods of the SCS TR-55, 1975, are developed primarily for application to urban watersheds and will be discussed in detail in Section 8.0 of this manual. The procedures described above, however, are also applicable to the estimation of peak flows for nonurban watersheds. In its discussion of hydrograph development, the SCS National Engineering Handbook, 1972, does give a peak flow formula, Equation (6-15) in this manual. The user is cautioned that this formula is for the peak flow of the unit hydrograph and is not applicable to the estimation of a peak design flood flow unless the design hydrograph is first developed in accordance with prescribed SCS procedures.

Some of the limitations of the SCS rainfall runoff method are closely associated with the manner in which initial abstractions and infiltration are taken into account. The initial abstraction is empirically determined to be 20 percent of the maximum storage, S, given by Equation (6-25). The basic assumption in deriving Equation (6-23) is that if an arithmetic plot is made of the accumulated rainfall excess against accumulated precipitation, then late in the storm, these two values approach one another; or $Q/P = 1$. However, at no earlier time, during the storm, does this equality hold. Morel-Seytoux and Verdin, 1981, have studied behavior of the SCS infiltration method in more detail. They have shown that the SCS method gives a monotonically decreasing infiltration curve only when the storm intensity is constant. For storms of variable intensity, the SCS infiltration curve is found to be discontinuous. They point out this may lead to unrealistic estimates of the rate of excess rainfall and therefore has a direct effect on the accuracy of the SCS synthetic unit hydrograph and any subsequent design hydrographs for ungaged watersheds.

Recognizing these potential limitations, Morel-Seytoux and Verdin, 1981, proposed an extension to the SCS Method which utilizes a physically based infiltration method. Their approach assumes an initial period in which all incident rainfall infiltrates. This initial period ends when the soil at the surface becomes saturated and ponding occurs. After ponding is complete, the infiltration capacity of the soil is assumed to follow a monotonically decreasing curve which asymptotically approaches the hydraulic conductivity of the soil at natural saturation.

Equations for post ponding time and time dependent monotonically decreasing infiltration capacity are presented for both constant and variable rainfall rates. These equations are functions of such soil properties as the soil moisture, rainfall intensity, the hydraulic conductivity of the soil at natural saturation and the effective capillary drive or wetting front suction. While the Morel-Seytoux and Verdin approach is theoretically more

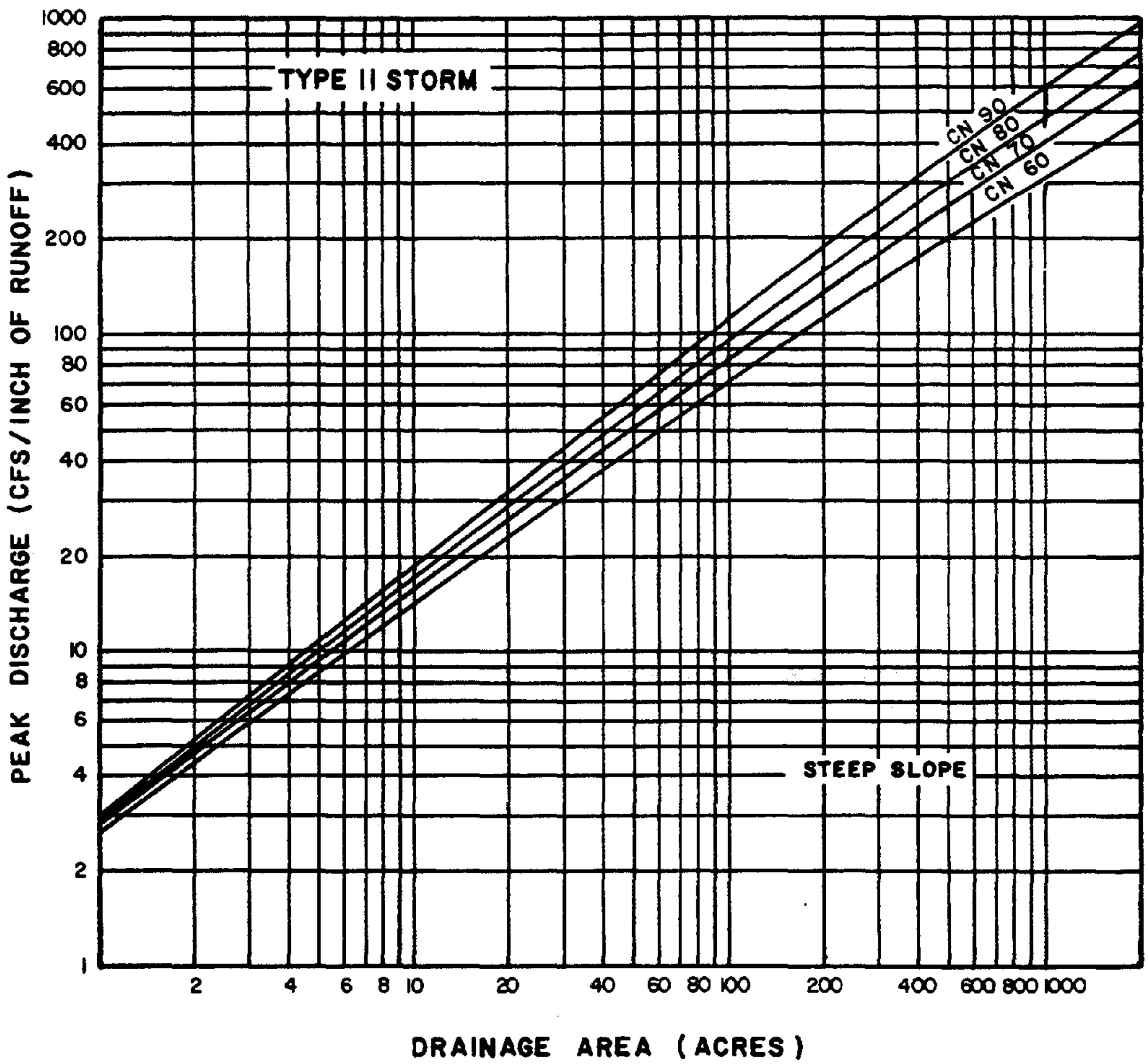



Figure 58. Peak Discharge as a Function of Drainage Area - Steep Slope

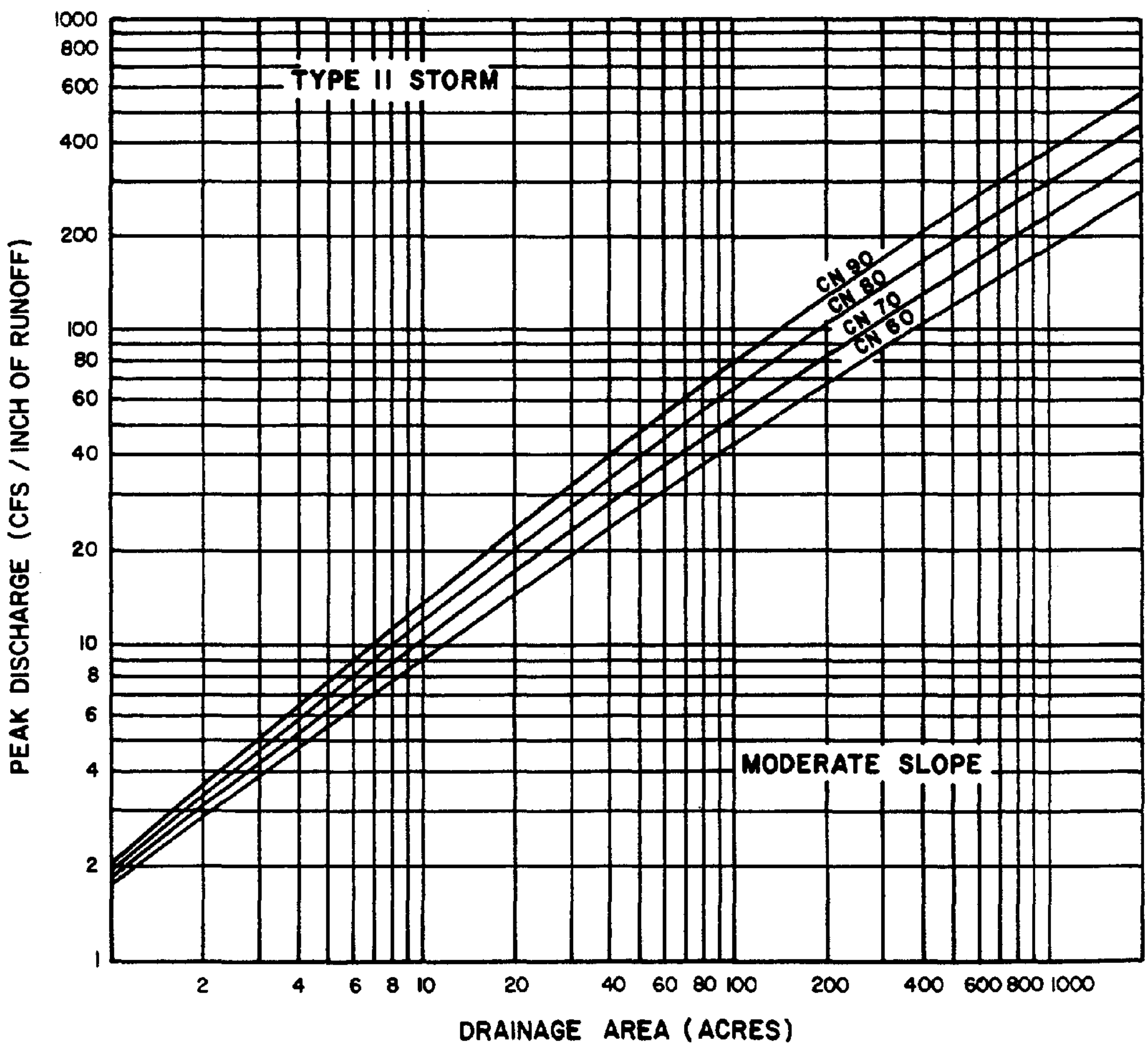



Figure 59. Peak Discharge as a Function of Drainage Area - Moderate Slope

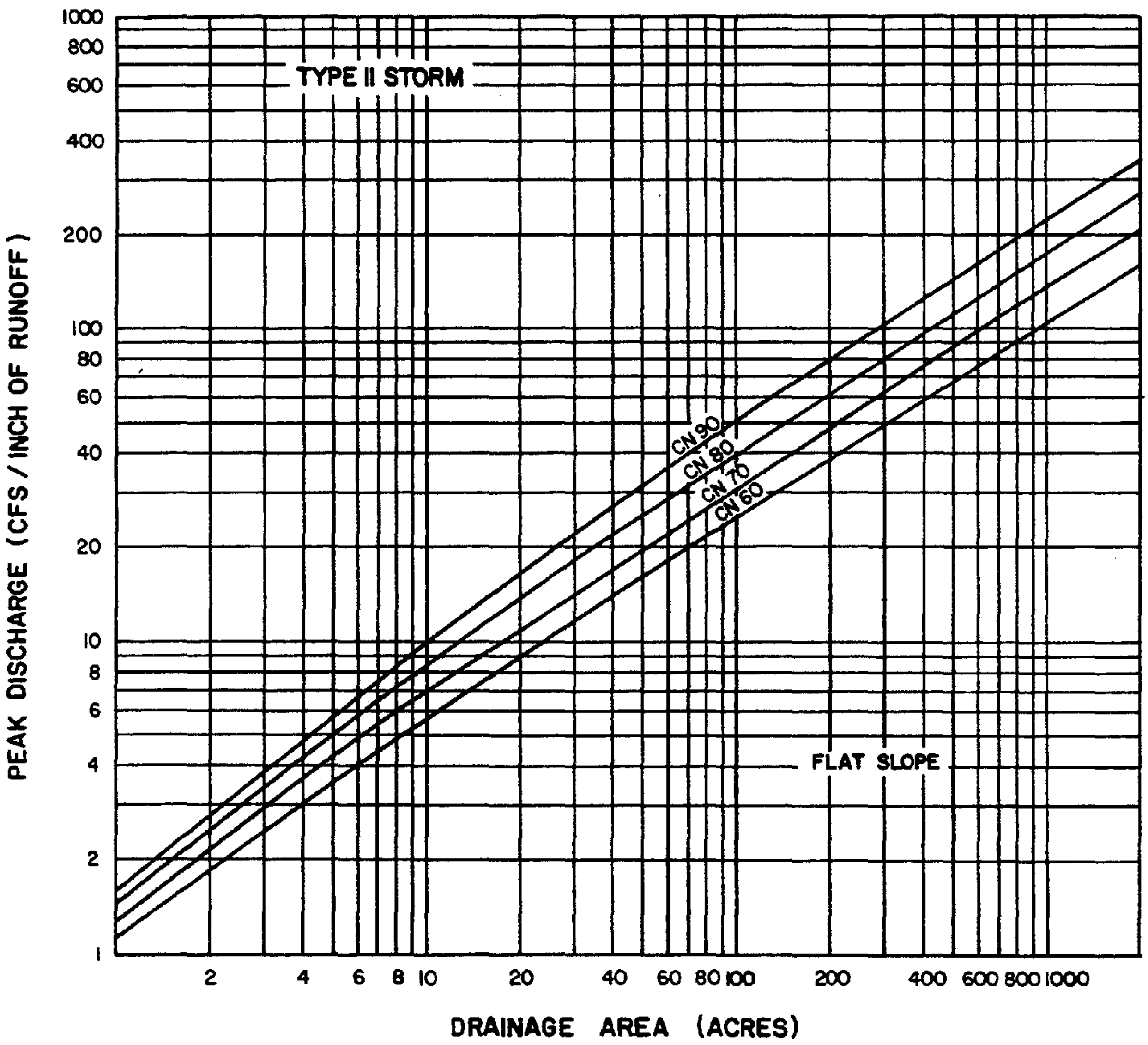



Figure 60. Peak Discharge as a Function of Drainage Area - Flat Slope

sound and overcomes some of the shortcomings of the SCS method, it requires the designer to estimate the soil parameters described above in order to utilize the method. Since most of these parameters are not readily available in standard references, they must be determined from rainfall-runoff data. This greatly limits the use of the infiltration approach in ungaged watersheds unless the needed data are available from a nearby similar watershed.

Because of the difficulty in acquiring the necessary soil data, a table of correspondence is established between the SCS curve number and the parameters necessary to implement the physical infiltration approach. This equivalence is based on the assumption that the amount of water abstracted from a constant intensity storm is the same whether calculated by the SCS method or by the physical infiltration approach. Regression analysis is used to generalize the results between the curve number and the hydraulic conductivity and sorptivity at field capacity for nine major soil types. (Only these two soil parameters are needed to determine the remaining inputs to the infiltration approach.) Since data from actual storms were used in developing the SCS curve numbers, an adjustment is provided by Morel-Seytoux and Verdin, 1981, to eliminate the bias resulting from the assumption of uniform storms in the development of the equivalence.

With the correspondence established between curve number and soil properties, the infiltration approach can be implemented as follows: A curve number for antecedent moisture condition II is determined for the watershed in a conventional manner from soil maps, land use and field inspection. The bias is then eliminated from the conventional CN value to obtain an adjusted value of CN to enter the Table of Correspondence from which the equivalent hydraulic conductivity and the storage suction factor can be obtained. With these two infiltration parameters, the remaining soil parameters can be determined and the infiltration method applied to the storm event and the pattern of excess rainfall computed. From this point, any suitable hydrograph method can be used to characterize the surface runoff.

## 6.4 Design Hydrographs

A design hydrograph is normally defined as the hydrograph associated with the design discharge and will have a specified frequency. Such hydrographs are usually the result of large or intense storms that vary in intensity and duration. The problem facing the designer is to select a storm with a pattern of intensity and duration which characterizes those storms which produce discharges of the desired magnitude.

If streamflow and precipitation records are available for a particular design site, the development of the design hydrograph is a straightforward procedure. Both unit hydrographs and unit storms can be determined from the data using the methods described in Secs. 6.1.3 and 6.1.4. Rainfall records can be readily analyzed to determine unit durations and the intensity which produces peak flows near the desired design discharges. If necessary, the unit hydrographs can be compounded and lagged to account for complex storms of different durations and varying intensities.

For basins without data, synthetic methods were described in Sec. 6.2 to develop unit hydrographs. These methods tend to be somewhat inflexible in the choice of unit storm duration since this value is determined by empirical relations in both the Snyder and SCS synthetic procedures. It is possible to enter these methods with a specified unit duration; however, the precipitation data must be available from which storms can be analyzed.

### 6.4.1 Design Storms

Several characteristics of design storms have already been defined in conjunction with construction of unit hydrographs. The design storms should be simple, individually occurring events with near uniform distribution over the period of rainfall excess. In addition, the storms should be uniform over the entire drainage area and be of sufficient intensity and duration to produce a measurable hydrograph.

#### 6.4.1.1 Design Storm from Rainfall-Runoff Data

The preferred method of determining an appropriate design storm is to analyze precipitation and runoff records for flood events of the magnitudes with which the designer is concerned. Records need not necessarily be for the specific drainage basin nor do they need to all be from the same watershed. Instead it is the characteristics of storms which produce large flood events that are sought. What are the durations and time variations of intensities? Are these storms characteristic of short, intense, convective storms or longer, more uniformly distributed cyclonic storms? Such information can help in generalizing the duration and intensity variation into a typical pattern to be used for design.

To illustrate the determination of a design storm, an example using three storms is presented as follows. With data from nearby gaged watersheds supplemented with simulated peak flows, a characteristic log-Pearson III distribution for watersheds on the order of eight square miles in Dallas County, Texas, has been determined as shown in Figure 61. A drainage structure is to be designed on Little Fossil Creek for a 25-year peak flow of 4530 CFS. It is further required to develop the hydrograph associated with this peak flow.

U.S. Geological Survey precipitation and runoff data were reviewed for the period 1975-1978 for 13 drainage basins in the county. Over 15 storms were found to produce peak flows on the order of 4000 to 5000 CFS. Some of the storms were rejected initially because the hydrographs contained multiple peaks and the storms were not isolated events. The remaining hydrographs were found to result from short duration (approximately 2 hours) convective or thunderstorms and longer duration (approximately 12 hours) cyclonic storms.

Upon further analysis of the rainfall distributions, it was found that intensities associated with the short duration thunderstorms were more

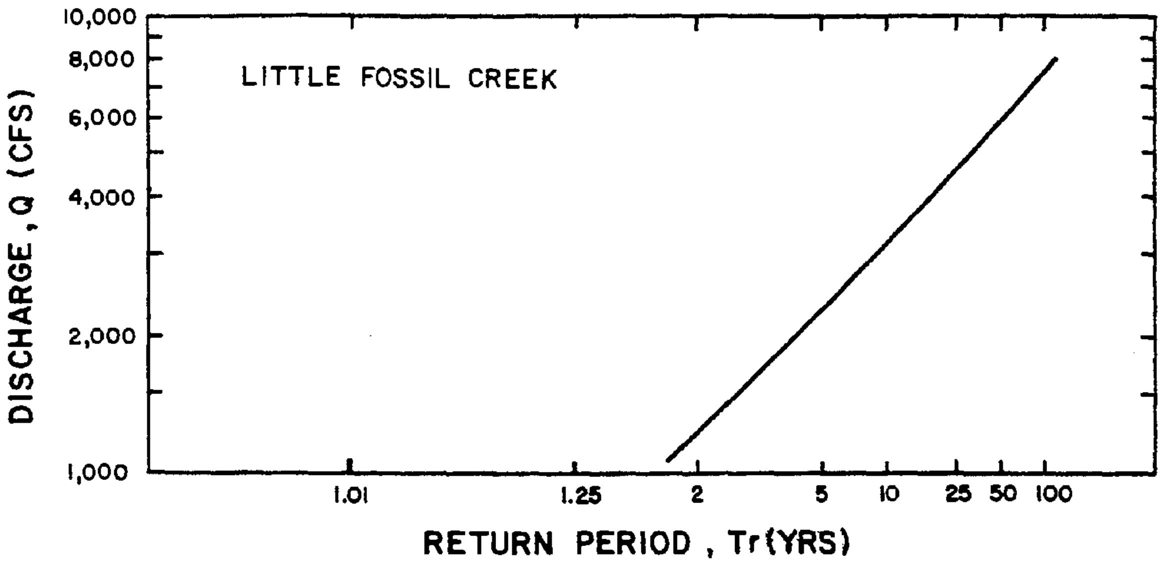



Figure 61. Frequency Analysis for Design Hydrograph Development

uniform and the storms were better defined. Three storms were selected from the USGS data (1975, 1978), the characteristics of which are summarized in Figures 62, 63, and 64.

From these data, rainfall intensity hyetographs with 15-minute intervals are plotted as shown in Figure 65a. (The intensity is determined from the slope of the curve of accumulated rainfall in inches).

Once the hyetograph for each storm has been developed, the next step is to determine how it is modified by all of the losses which transform rainfall into runoff. As stated many times, this is a very complex and ill understood process. Consequently, simplifying assumptions are used to facilitate analysis. Using the technique of accounting for losses in two phases namely initial abstractions and infiltration, as presented in Section 6.1.3.4, the storm hyetographs are converted into excess rainfall hyetographs.

The initial abstractions are the volumes of rainfall prior to the start of direct runoff. The remaining infiltration is determined by the $\Phi$ index. The direct runoff volume is taken as the accumulated runoff. The $\Phi$ index and excess rainfall hyetographs are shown in Figure 65b.

For these three storms the unit duration is 1 hour and the average intensity of excess rainfall is about 1.33 inches/hour. In other words a design storm with a unit duration of 1-hour and a rainfall excess of 1.33 inches should produce a design hydrograph with a peak flow in the range of 4000 to 5000 CFS.

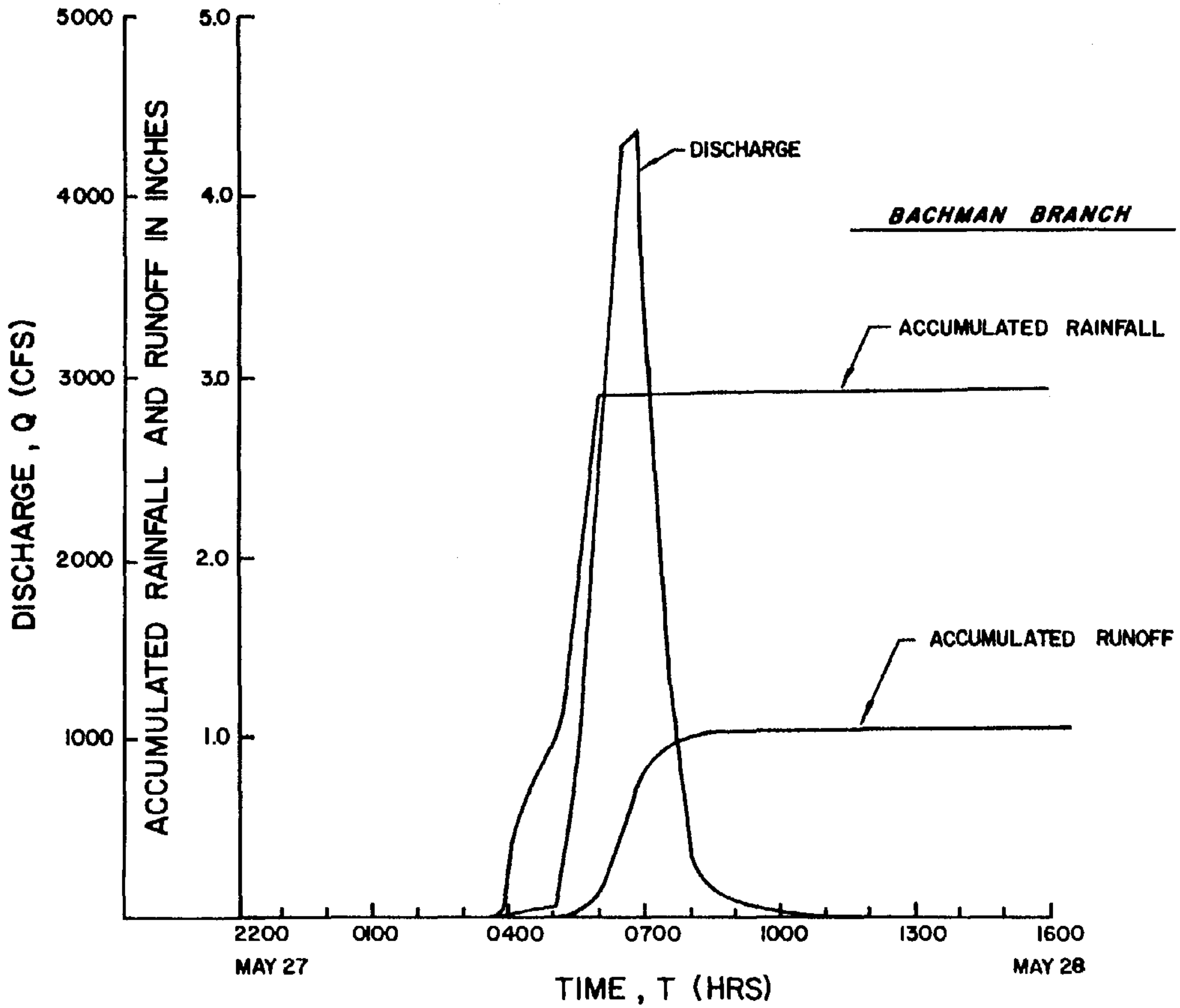



Figure 62. Precipitation and Runoff Data for Bachman Branch, Storm of May 27-28, 1978

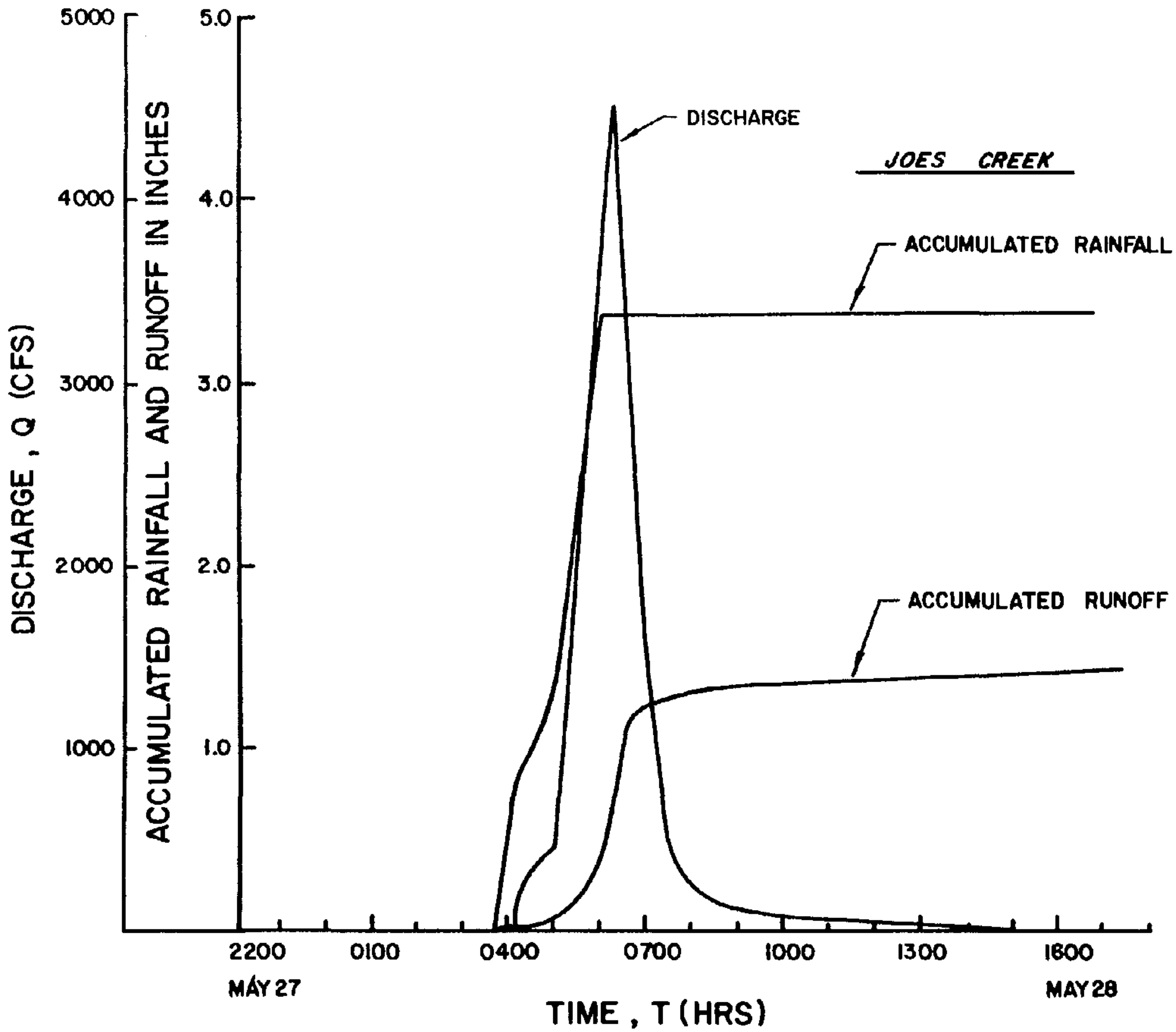



Figure 63. Precipitation and Runoff Data for Joes Creek, Storm of May 27-28, 1978

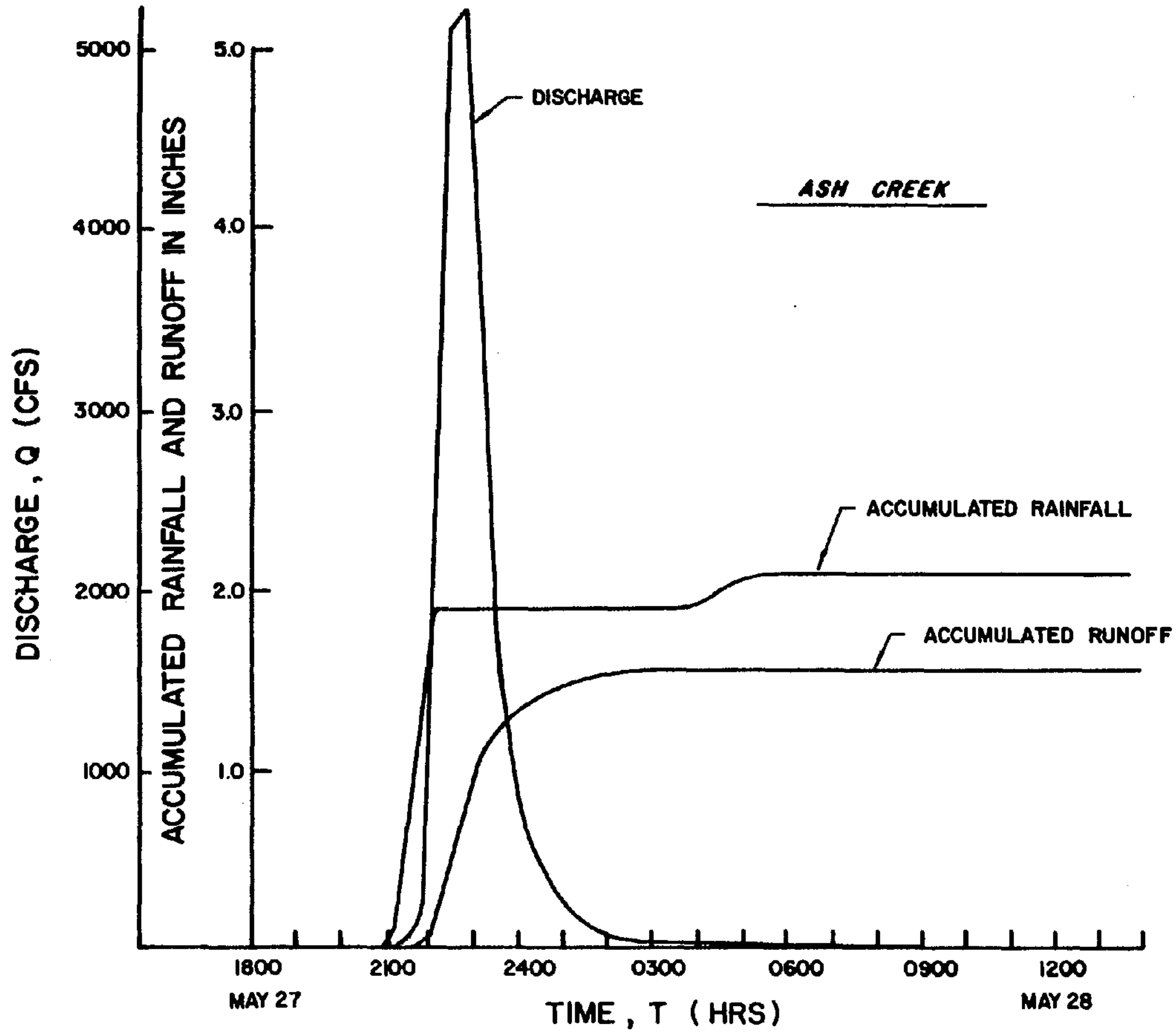



Figure 64. Precipitation and Runoff Data for Ash Creek, Storm of May 27-28, 1975

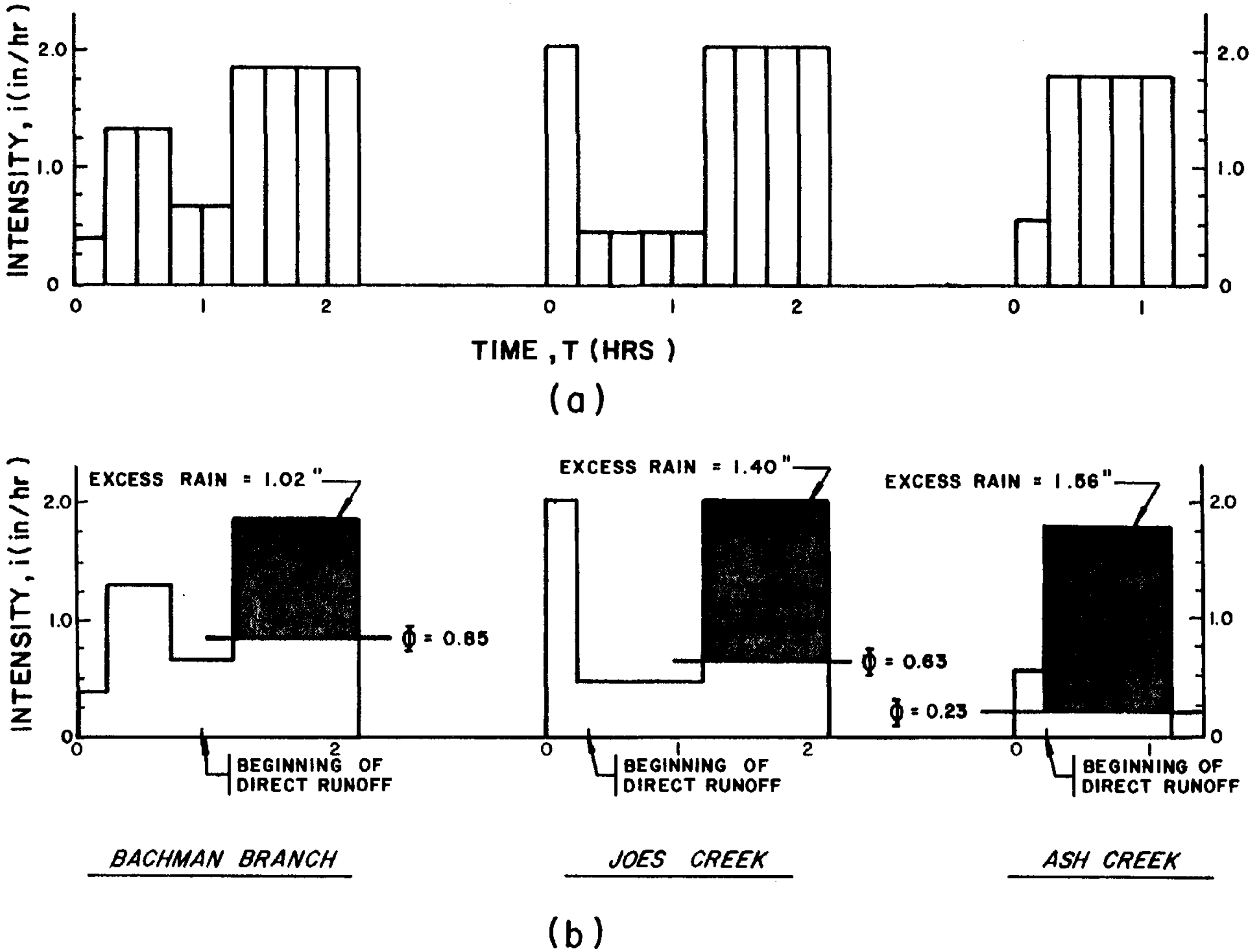



Figure 65. Precipitation and Excess Rainfall Hyetographs for Bachman Branch and Joes and Ash Creeks

In the absence of runoff data, it becomes necessary to rely totally on synthetic unit hydrograph methods to determine design hydrographs. The techniques permit the unit storm duration to be computed empirically as a reference so that the peak flow can be positioned in time through the concept of lag. The intensity of the unit storm is not usually computed; however it can be readily determined knowing its duration and that the volume of runoff from the drainage area is the 1-inch under the synthetic unit hydrograph.

Before selecting a design storm, it is especially important to compare the duration of the unit storm with the durations of storms typical of the area, i.e. short intense thunderstorms or long duration, moderate to low intensity cyclonic storms. If there are large variations between actual storms and the unit storm duration, the synthetic unit hydrograph should be lagged or compounded to obtain a more realistic unit hydrograph. The intensity of the design storm can then be determined from either an analysis of rainfall data or from intensity-duration-frequency curves given by the U.S. Weather Bureau after an appropriate deletion of initial abstractions and infiltration.

#### 6.4.1.2 Design Storm by Triangular Hyetograph

In 1983, Yen, B.C. and Chow, V.T. developed a method for approximating a design storm hyetograph by a triangular distribution applicable to watersheds smaller than 20 square miles (50 $km^2$). Their approach recognizes that a rainfall hyetograph, being a geometric figure, can be characterized by its moment with respect to the beginning of precipitation. Since no two rainstorms are alike, the statistical means of the moments of many rainstorms indicate the average characteristics of an expected storm.

The triangular representation used by Yen and Chow, 1983, is illustrated in Figure 66. The important geometric characteristics are the peak intensity, h, the time to peak, a, and the time dimension, b, equal to the duration $t_d$, minus the time to peak intensity. The hyetograph is then normalized as shown in Figure 67 using the duration of the storm, $t_d$, and the total depth of rainfall, D, in inches. Once the normalized value of the time to peak is known, the remaining values of the triangular hyetograph can be calculated from geometrics. The depth of rainfall depends on the duration and return period and typically would be specified by design practice or determined through a risk analysis or other economic evaluation. The duration of the design storm would be determined by the time of concentration so that the entire watershed would be contributing to the flow at the point of interest.

Yen and Chow, 1983, then analyzed 293,946 storms from 222 National Weather Stations (NWS) and 13 Agricultural Research Service (ARS) raingage stations to determine the statistical values of the normalized hyetograph parameters. They present the results in a series of maps with point values of the normalized time to peak intensity reported throughout the country for the NWS storms with durations of 2, 3, 4 and 5 hours and for durations of 10 to 20 minutes and 1, 2 and 4 hours for the 13 ARS raingage stations. A national map of the peak rain time of the triangular hyetograph is also presented which is suitable for use in highway design for heavy rainstorms.

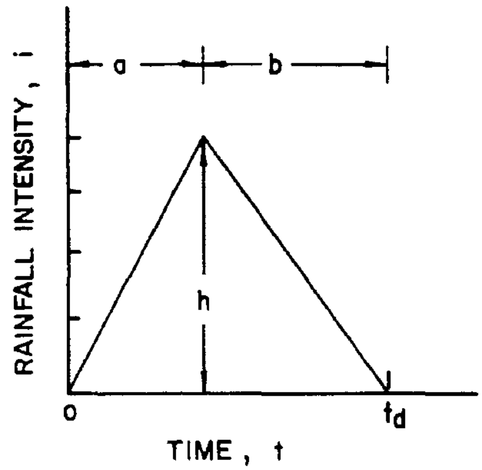



Figure 66. Triangular Hyetograph

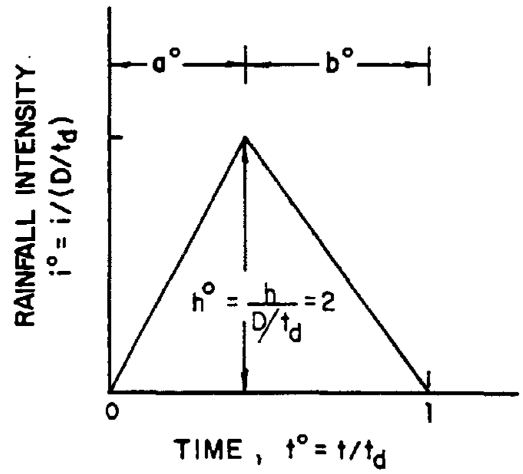



Figure 67. Normalized Triangular Hyetograph

6.4.2 Design Hydrograph by Transposition

Often the designer is confronted with the problem where streamflow and rainfall data are not available for a particular site but may exist at points upstream or in adjacent or nearby watersheds. If a design hydrograph can be developed at an upstream point in the same watershed, the procedures described in Section 7.1 can be used to route the design hydrograph to the point of interest. When the data for developing unit hydrographs exist in nearby hydrologically similar watersheds, the transposition method described in Sec. 6.2.4 can be used to obtain a design hydrograph.

To illustrate the transposition method, unit hydrographs can now be constructed for each of the three drainage areas for which design storms were developed above. Using the methods described in Sec. 6.1.3.3, the three unit hydrographs are as shown in Figure 68. Considering the peak flow, time to peak and runoff duration, an average unit hydrograph is obtained with the transposition procedure described in Sec. 6.2.4. The lag time, or the time from the midpoint of excess rainfall to the peak of the hydrograph, is determined from Figures 62, 63, 64 and 65b. These values together with L, $L_{ca}$, A and Y for each of the three watersheds and for Little Fossil Creek are summarized as follows.

| | L (mi) | $L_{ca}$ (mi) | Y (%) | Lag-Hrs | Drainage area (sq mi) |
|---|---|---|---|---|---|
| Bachman Branch | 5.9 | 3.0 | 0.60 | 1.27 | 10.00 |
| Joes Creek | 5.4 | 2.9 | 0.56 | 0.80 | 7.51 |
| Ash Creek | 4.2 | 2.5 | 0.65 | 0.95 | 6.92 |
| Little Fossil Creek | 10.1 | 3.5 | 0.40 | | 12.30 |

If the lag is plotted against $(LL_{ca}/Y^{0.5})$ on full logarithmic graph paper for three watersheds with unit hydrographs, the values of C and k can be estimated and Equation (6-22) becomes

$$\text{Lag} = 0.53\left(\frac{LL_{CA}}{Y^{0.5}}\right)^{0.23}$$

With L, $L_{ca}$ and Y also known for Little Fossil Creek, the lag time for the transposed unit hydrograph can be calculated as

$$\text{Lag} = 0.53\left(\frac{(10.1)(3.5)}{(0.40)^{0.5}}\right)^{0.23} = 1.34 \text{ HRS}$$

Actually, it would be preferable to use more than three watersheds for the determination of the constant and exponent in Equation (6-22).

Similarly, if the peak flows of the unit hydrographs are plotted against the drainage area, the following equation is obtained

$$Q_p = 2248\, A^{0.187}$$

For a drainage area of 12.3 sq mi, the peak of the unit hydrograph for Little Fossil Creek is 3594 CFS (101.8 CMS). This value may be in error because of the difficulty in establishing the relation between $Q_p$ and A with only three points. However, the method of transposition is illustrated and with the peak flow and lag defined, the unit hydrograph for Little Fossil Creek can be constructed as shown in Figure 68. The shape of this unit hydrograph is the average shape of the three unit hydrographs used in its development and its volume has been adjusted to 1-inch of runoff.

The design hydrograph is then determined by multiplying the average unit hydrograph ordinates by the average excess rainfall of the design storm as illustrated in Figure 69. (The unit hydrograph could have also been determined by the synthetic methods described in Section 6.2).

It is probable that the peak discharge of the resulting design hydrograph will not agree with the peak discharge determined from the frequency

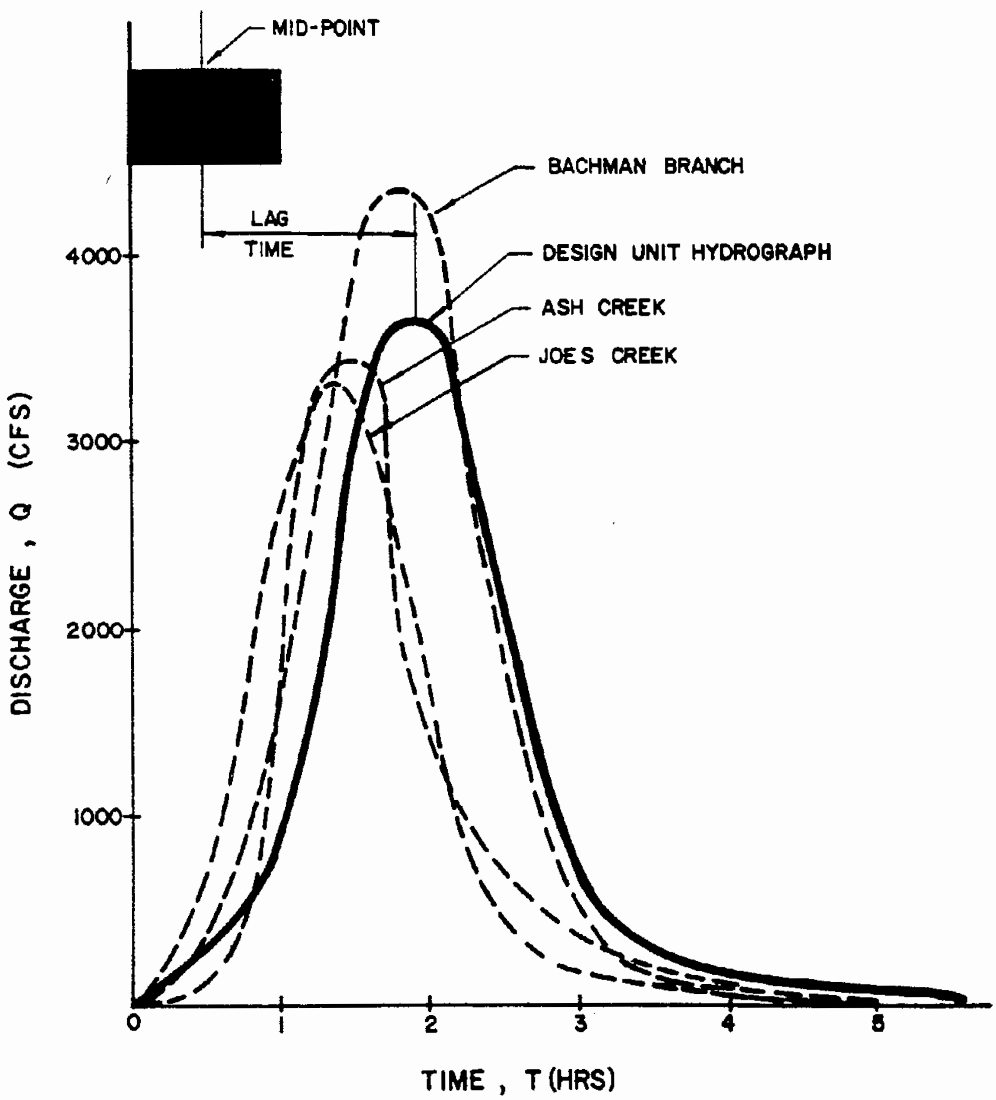



Figure 68. 1-Hour Unit Hydrographs for Bachman Branch and Joes and Ash Creeks

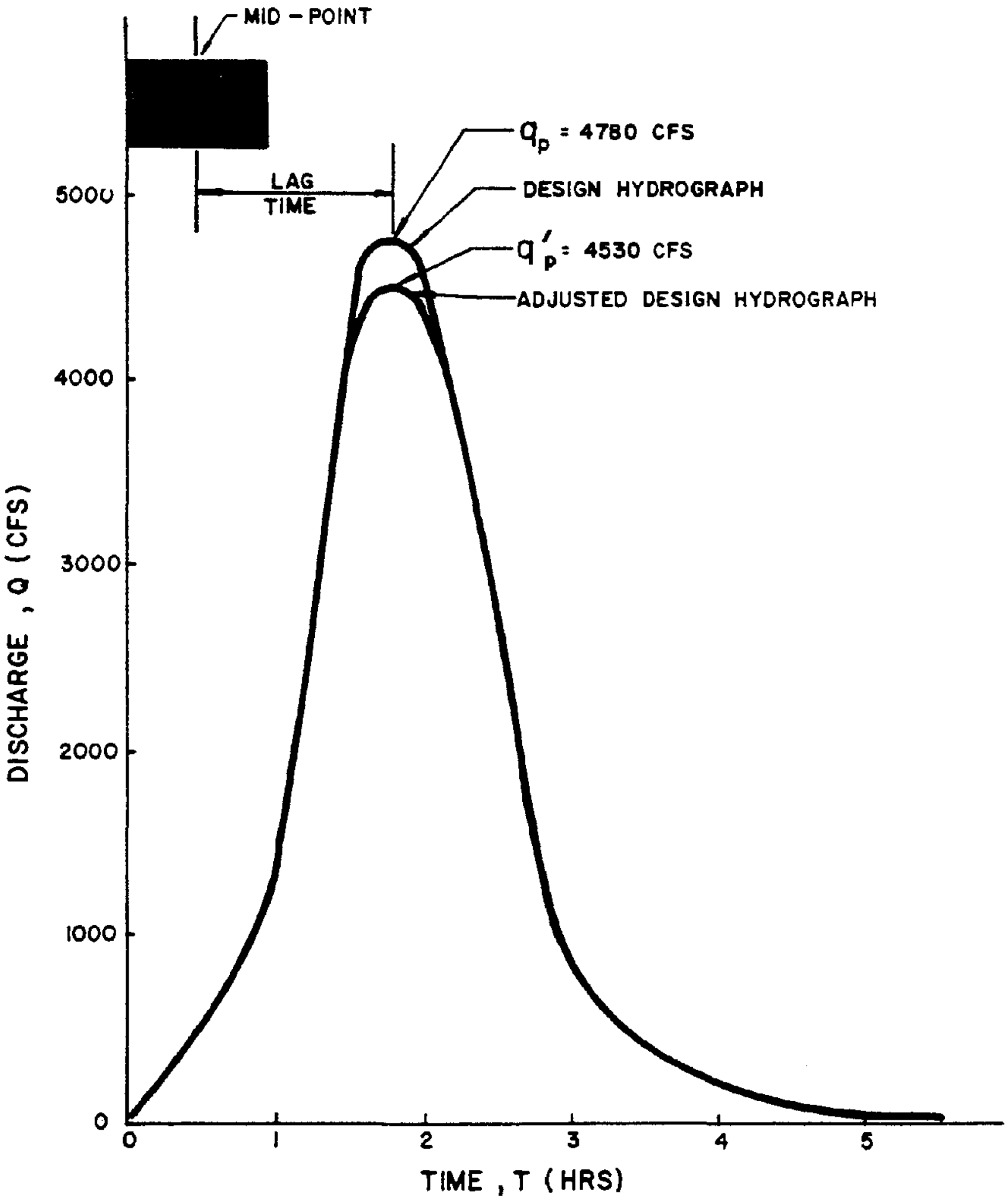



Figure 69. Design Hydrograph Determined from Storms on Bachman Branch and Joes and Ash Creeks

analysis. The designer can adjust the design hydrograph by multiplying the hydrograph ordinates by the ratio of $q'_p/q_p$ where $q'_p$ is the desired peak flow at the specified return period. In the above example, $q'_p/q_p$ = (4530/4780) = 0.95. The adjusted hydrograph, also shown in Figure 69, will have a peak flow equal to the desired discharge and will have a realistic hydrograph shape.

### 6.4.3 Design Hydrograph by SCS Methods

The Soil Conservation Service has developed an approach to obtain design hydrographs for proportioning earth dams and their spillways. Although the emphasis is primarily for storage and flood protection, the methods have application to a wide variety of design problems associated with channels, channel works and control structures. This design hydrograph is referred to as the Primary Spillway Hydrograph or PSH and the associated mass curve as the PSMC. The techniques for its development and several illustrative examples are discussed in the SCS Handbook, 1972.

Four methods are listed as satisfactory for the determination of runoff. They are:

1. runoff Curve Number procedure using rainfall data and watershed characteristics,
2. runoff volume maps convering specific areas of the United States,
3. regionalization and transposition of volume-duration-frequency analysis, and
4. local streamflow data.

Only the first two methods are described by the SCS since in the latter two, each situation is a special case depending on local data and standard procedures have not been developed.

### 6.4.4 Runoff Curve Number Procedure

Before direct runoff can be estimated, this procedure requires rainfall data for durations of 1 and 10 days. These data can be obtained from appropriate Technical Papers of the U.S. Weather Bureau, (T.P.-40, 42, 43 and 47 for durations up to 1 day and T.P.-49, 51, 52 and 53 for durations from 2 to 10 days). If the drainage area is less than 10 square miles, no adjustment to rainfall is made. If the drainage area is over 10 square miles, the rainfall amounts are adjusted by area point ratios given in Table 42.

The runoff curve number (CN) for the watershed is determined from Table 37 for an antecedent moisture condition II and applies to the 1-day duration. If the 100-year frequency 10-day duration is less than 6 inches, the CN value for the 10-day duration is the same as that for the 1-day duration. If it exceeds 6 inches, the CN value for the 10-day duration is taken from Table 43.

Table 42. Ratios for Areal Adjustment of Rainfall Amount

| Area | Area/point ratio for 1 day | Area/point ratio for 10 days | Area | Area/point ratio for 1 day | Area/point ratio for 10 days |
|---|---|---|---|---|---|
| sq mi | | | sq mi | | |
| 10 or less | 1.000 | 1.000 | 80 | 0.937 | 0.968 |
| 15 | .978 | .991 | 100 | .932 | .966 |
| 20 | .969 | .986 | 120 | .928 | .964 |
| 25 | .964 | .983 | 140 | .925 | .962 |
| 30 | .960 | .981 | 160 | .922 | .961 |
| 35 | .957 | .979 | 180 | .920 | .960 |
| 40 | .953 | .977 | 200 | .918 | .959 |
| 50 | .948 | .974 | 250 | .914 | .957 |
| 60 | .944 | .972 | 300 | .911 | .956 |
| 70 | .940 | .970 | 400 | .910 | .955 |

from SCS, 1972

Table 43. Ten-Day Runoff Curve Numbers for 100-Year, 10-Day Point Rainfall Equal to or Greater than 6 Inches

| Runoff Curve Numbers for: 1 day | 10 days | 1 day | 10 days | 1 day | 10 days |
|---|---|---|---|---|---|
| 100 | 100 | 80 | 65 | 60 | 41 |
| 99 | 98 | 79 | 64 | 59 | 40 |
| 98 | 96 | 78 | 62 | 58 | 39 |
| 97 | 94 | 77 | 61 | 57 | 38 |
| 96 | 92 | 76 | 60 | 56 | 37 |
| 95 | 90 | 75 | 58 | 55 | 36 |
| 94 | 88 | 74 | 57 | 54 | 35 |
| 93 | 86 | 73 | 56 | 53 | 34 |
| 92 | 84 | 72 | 54 | 52 | 33 |
| 91 | 82 | 71 | 53 | 51 | 32 |
| 90 | 81 | 70 | 52 | 50 | 32 |
| 89 | 79 | 69 | 51 | 49 | 31 |
| 88 | 77 | 68 | 50 | 48 | 30 |
| 87 | 76 | 67 | 48 | 47 | 29 |
| 86 | 74 | 66 | 47 | 46 | 28 |
| 85 | 72 | 65 | 46 | 45 | 27 |
| 84 | 71 | 64 | 45 | 44 | 27 |
| 83 | 69 | 63 | 44 | 43 | 26 |
| 82 | 68 | 62 | 43 | 42 | 25 |
| 81 | 66 | 61 | 42 | 41 | 24 |

from SCS, 1972

This SCS design hydrograph procedure then requires the determination of a climatic index defined as

$$C_i = \frac{100 P_a}{(T_a)^2} \qquad (6\text{-}27)$$

where $C_i$ is the climatic index, $P_a$ is the average annual precipitation in inches, and $T_a$ is the average annual temperature in °F. Average precipitation and temperature data can be obtained from such U.S. Weather Bureau publications as Climatological Data, Climatic Summary of the United States and Climates of the States. Although channel losses due to influent streams can be determined from local streamflow data, the climatic index can be used to make this adjustment. Table 44 summarizes channel loss factors for the reduction of direct runoff as a function of the climatic index and drainage area.

Table 44. Channel-Loss Factors for Reduction of Direct Runoff

| Drainage area | Climatic Index, $C_i$ | | | | | | |
|---|---|---|---|---|---|---|---|
| | 1.0 | 0.9 | 0.8 | 0.7 | 0.6 | 0.5 | 0.4 or less |
| sq mi | | | | | | | |
| 1 or less | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 2 | 1.00 | .99 | .97 | .96 | .93 | .90 | .83 |
| 3 | 1.00 | .98 | .96 | .92 | .89 | .84 | .79 |
| 4 | 1.00 | .97 | .94 | .91 | .86 | .81 | .74 |
| 5 | 1.00 | .97 | .93 | .90 | .84 | .78 | .70 |
| 6 | 1.00 | .96 | .93 | .88 | .82 | .76 | .68 |
| 7 | 1.00 | .96 | .92 | .87 | .81 | .74 | .66 |
| 8 | 1.00 | .96 | .92 | .86 | .80 | .73 | .64 |
| 9 | 1.00 | .95 | .91 | .85 | .79 | .72 | .62 |
| 10 | 1.00 | .95 | .90 | .84 | .78 | .70 | .60 |
| 15 | 1.00 | .94 | .89 | .82 | .75 | .67 | .56 |
| 20 | 1.00 | .94 | .88 | .80 | .72 | .63 | .52 |
| 30 | 1.00 | .93 | .86 | .78 | .69 | .60 | .48 |
| 40 | 1.00 | .92 | .85 | .76 | .67 | .57 | .45 |
| 50 | 1.00 | .92 | .84 | .75 | .66 | .55 | .43 |
| 60 | 1.00 | .92 | .84 | .74 | .64 | .54 | .41 |
| 70 | 1.00 | .92 | .83 | .73 | .63 | .52 | .40 |
| 80 | 1.00 | .92 | .82 | .72 | .62 | .51 | .38 |
| 100 | 1.00 | .91 | .81 | .71 | .61 | .50 | .37 |
| 150 | 1.00 | .90 | .80 | .69 | .58 | .47 | .34 |
| 200 | 1.00 | .90 | .79 | .68 | .56 | .45 | .32 |
| 300 | 1.00 | .89 | .78 | .65 | .54 | .42 | .29 |
| 400 | 1.00 | .88 | .76 | .64 | .52 | .40 | .27 |

from SCS, 1972

A quick return flow (QRF) is then defined in the SCS procedure as that flow which persists beyond the 10-day hydrograph duration. The quick return flow is not as important to highway drainage projects as it is for storage and earth-filled dam design. The (QRF) is considered to consist of infiltration that reappears as surface runoff and delayed drainage from swamps, marshes, potholes and snowpack.

Throughout the discussion of design hydrograph, the SCS Handbook, 1972, emphasizes the purpose of the procedure is to develop a safe design rather than to reproduce actual or historical floods. It is primarily for this reason that the various adjustments described above are recommended and that combinations of channel losses, quick return flow and upstream releases are included in the analysis. It would also be appropriate to include upstream releases when applying this SCS method to highway design if it is determined that such releases would affect the peak flow.

In a manner analogous to that for the SCS synthetic unit hydrograph method discussed in Section 6.2.3, the design hydrograph is proportioned from a standard series of PSH and PSMC tabulations provided in the SCS Handbook, 1972. These tabulations, comprising 22 pages, summarize time, rate and mass for design hydrographs (PSH) and mass curves (PSMC) for times of concentration, $T_c$, ranging from 1.5 to 72 hours and $Q_1/Q_{10}$ ratios of 0.2 to 0.9 for each value of $T_c$, a total of 112 sets of hydrograph coordinates. Table 45 is typical of one page of this tabulation. The various sets of coordinates are also identified by Serial Numbers which are readily obtained from a table in the SCS Handbook, 1972, which gives the Serial Number as a function of $T_c$ and $Q_1/Q_{10}$.

To illustrate the development of a design hydrograph by this method, the following example is taken directly from the SCS Handbook, 1972.

> It is desired to develop a 50-year design hydrograph for a 15.0 square mile drainage area which has an average annual precipitation of 22.8 inches and an average annual temperature of 61.5° F. The runoff curve number for the watershed is 80 and the time of concentration has been estimated at 7.1 hours.

1. For the location of this watershed, the 50-year frequency, 1-day and 10-day rainfall amounts have been determined from USWB, TP-40 and TP-49, respectively as

   1-day duration = 6.8 in

   10-day duration = 11.0 in

2. Since the drainage area is greater than 10 square miles, the areal adjustments for the rainfall amounts are determined from Table 42 as 0.978 for the 1-day duration and 0.991 for the 10-day duration. The adjusted rainfalls are

1-day duration: 0.978 (6.8) = 6.65 in

10-day duration: 0.991 (11.0) = 10.90 in

3. From Table 43, the CN value for the 10-day duration is 65 given that the 1-day duration CN is 80.

4. The direct runoffs for the 1- and 10-day durations can be determined from either Table 41 or Figure 56a. Using Figure 56a, the direct runoffs are

   1-day duration, CN = 80, Precipitation = 6.65 inches
   Direct Runoff = 4.37 inches

   10-day duration, CN = 65, Precipitation = 10.90 inches
   Direct Runoff = 6.34 inches

5. The climatic index is computed from the given data and Equation (6-27) as

$$C_i = 100 \frac{P_a}{(T_a)^2} = 100 \frac{(22.8)}{(61.5)^2} = 0.603$$

   and the net runoff is obtained by adjusting direct runoff by the channel loss factors in Table 44. For $C_i = 0.603$ and a drainage area of 15.0 square miles, the channel loss factor is 0.75 and the net runoffs are

   1-day duration: 4.37 (0.75) = 3.28 inches

   10-day duration: 6.34 (0.75) = 4.76 inches

6. The $Q_1/Q_{10}$ is computed as

$$Q_1/Q_{10} = 3.28/4.76 = 0.689$$

7. With $Q_1/Q_{10} = 0.689$ and a time concentration of 7.1 hours, the nearest PSH tabulation is found which will correspond to Serial Number 22 in Table 45. The product of $Q_{10}A$ is first determined as (4.76)(15.0) = 71.4, and the design hydrograph ordinates are shown in the following summary table.

   The resulting design hydrograph is also plotted in Figure 70.

Table 45. Time, Rate and Mass Tabulations for Design Hydrographs (PSH) and Mass Curves (PSMC)

$T_c$ = 6 hours

| Serial No. : | 21 | | 22 | | 23 | | 24 | |
|---|---|---|---|---|---|---|---|---|
| $Q_1/Q_{10}$ : | 0.6 | | 0.7 | | 0.8 | | 0.9 | |
| Time | PSH | PSMC | PSH | PSMC | PSH | PSMC | PSH | PSMC |
| days | $cfs/AQ_{10}$ | $Q/Q_{10}$ | $cfs/AQ_{10}$ | $Q/Q_{10}$ | $cfs/AQ_{10}$ | $Q/Q_{10}$ | $cfs/AQ_{10}$ | $Q/Q_{10}$ |
| .0 | .000 | .000 | .000 | .0000 | .000 | .0000 | .000 | .0000 |
| .2 | .346 | .0010 | .231 | .0007 | .130 | .0004 | .058 | .0002 |
| .5 | .621 | .0068 | .418 | .0045 | .254 | .0026 | .124 | .0012 |
| 1.0 | .719 | .0193 | .535 | .0135 | .302 | .0079 | .160 | .0039 |
| 2.0 | .881 | .0486 | .610 | .0340 | .412 | .0218 | .194 | .0102 |
| 3.0 | 1.167 | .0865 | .837 | .0609 | .566 | .0398 | .274 | .0188 |
| 3.6 | 1.518 | .1163 | 1.123 | .0827 | .708 | .0536 | .395 | .0262 |
| 4.0 | 1.934 | .1428 | 1.398 | .1019 | 1.004 | .0668 | .510 | .0331 |
| 4.3 | 2.527 | .1666 | 1.932 | .1196 | 1.489 | .0804 | .784 | .0401 |
| 4.6 | 3.539 | .1997 | 2.865 | .1464 | 1.961 | .0987 | .999 | .0500 |
| 4.8 | 4.747 | .2295 | 3.973 | .1709 | 2.887 | .1161 | 1.555 | .0591 |
| 4.9 | 6.335 | .2499 | 5.461 | .1883 | 4.056 | .1289 | 2.255 | .0661 |
| 5.0 | 22.276 | .3026 | 27.118 | .2482 | 32.166 | .1955 | 37.622 | .1394 |
| 5.1 | 42.826 | .4225 | 55.278 | .3998 | 69.093 | .3817 | 84.295 | .3634 |
| 5.2 | 33.204 | .5625 | 41.011 | .5770 | 49.241 | .5993 | 57.738 | .6245 |
| 5.3 | 20.462 | .6613 | 23.735 | .6961 | 26.833 | .7392 | 29.654 | .7851 |
| 5.4 | 12.851 | .7226 | 13.975 | .7655 | 14.846 | .8159 | 15.379 | .8679 |
| 5.5 | 8.521 | .7619 | 8.668 | .8072 | 8.572 | .8589 | 8.194 | .9112 |
| 5.6 | 5.896 | .7885 | 5.638 | .8335 | 5.120 | .8841 | 4.424 | .9344 |
| 5.8 | 3.326 | .8212 | 2.818 | .8634 | 2.199 | .9096 | 1.490 | .9546 |
| 6.0 | 2.389 | .8417 | 1.859 | .8798 | 1.326 | .9216 | .680 | .9616 |
| 6.5 | 1.655 | .8764 | 1.360 | .9078 | .931 | .9409 | .438 | .9711 |
| 7.0 | 1.322 | .9031 | 1.002 | .9290 | .666 | .9551 | .327 | .9779 |
| 7.5 | 1.085 | .9249 | .804 | .9453 | .525 | .9658 | .253 | .9832 |
| 8.0 | .918 | .9431 | .687 | .9588 | .415 | .9743 | .221 | .9875 |
| 9.0 | .718 | .9730 | .533 | .9812 | .305 | .9880 | .165 | .9944 |
| 9.9 | .586 | .9952 | .416 | .9966 | .271 | .9978 | .129 | .9990 |
| 10.1 | .272 | .9986 | .194 | .9990 | .122 | .9988 | .057 | .9997 |
| 10.3 | .062 | .9997 | .044 | .9998 | .028 | .9999 | .013 | .9999 |
| 10.8 | .000 | 1.0000 | .000 | 1.0000 | .000 | 1.0000 | .000 | 1.0000 |

| Time | $\frac{CFS}{A\ Q_{10}}$ | Design Hydrograph Ordinates |
|---|---|---|
| days | csm/inch | CFS |
| .0 | .000 | 0 |
| .2 | .231 | 16 |
| .5 | .418 | 30 |
| 1.0 | .535 | 38 |
| 2.0 | .610 | 44 |
| 3.0 | .837 | 60 |
| 3.6 | 1.123 | 80 |
| 4.0 | 1.398 | 100 |
| 4.3 | 1.932 | 138 |
| 4.6 | 2.865 | 204 |
| 4.8 | 3.973 | 284 |
| 4.9 | 5.461 | 390 |
| 5.0 | 27.118 | 1936 |
| 5.1 | 55.278 | 3947 |
| 5.2 | 41.011 | 2928 |
| 5.3 | 23.735 | 1695 |
| 5.4 | 13.975 | 998 |
| 5.5 | 8.668 | 619 |
| 5.6 | 5.638 | 402 |
| 5.8 | 2.818 | 201 |
| 6.0 | 1.859 | 133 |
| 6.5 | 1.859 | 97 |
| 7.0 | 1.002 | 72 |
| 7.5 | .804 | 57 |
| 8.0 | .687 | 59 |
| 9.0 | .533 | 38 |
| 9.9 | .416 | 30 |
| 10.1 | .194 | 14 |
| 10.3 | .044 | 3 |
| 10.8 | .000 | 0 |

### 6.4.5 Flood Hydrographs by Program XSRAIN

In Sec. 6.3, an extension to the SCS rainfall-runoff methodology by Morel-Seytoux and Verdin was described which utilized physical infiltration equations as an alternate for determining initial abstractions, infiltration and excess rainfall. In 1981, Verdin and Morel-Seytoux reported on a FORTRAN IV program entitled XSRAIN to calculate flood hydrographs for ungaged watersheds. The program utilizes the SCS Curve Number, CN, to characterize soil and land use types, Table 37, and the SCS dimensionless unit hydrograph and mass curves, Table 39, to route the excess rainfall determined by the infiltration approach to obtain the runoff hydrograph.

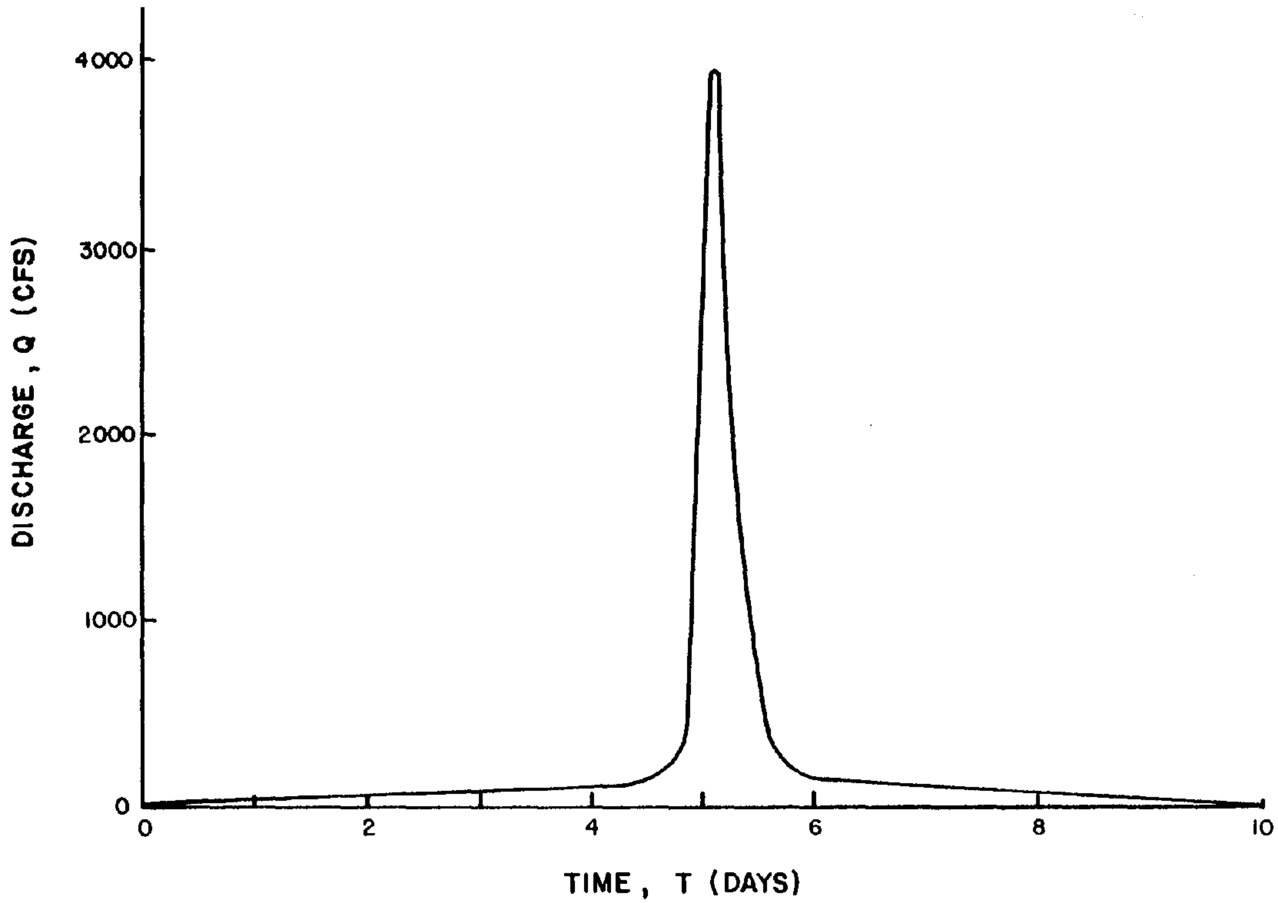



Figure 70. SCS 50-Year Frequency Design Hydrograph

The program, XSRAIN, does not use SCS equations (or the values reported in Table 41 and Figures 56a and 56b). Instead the program permits user specified variable intensity rainfalls with abstractions based on infiltration equations. The distributions of rainfall used in XSRAIN are those identified by Huff, 1967, in which storms in Central Illinois are categorized according to whether the rainfall occurs in the first, second, third or fourth quartile of the storm duration. As alternates, the designer may specify the rainfall distribution "as is" or rearrange the distribution according to the Corps of Engineers' "balanced hyetograph" wherein the maximum rainfall is the central element, the second highest is placed just before the maximum, the third highest just after the maximum, etc. Regardless of which rainfall distribution is selected, the user must specify the cumulative depth of rainfall and the storm duration as determined from design needs.

The infiltration is calculated from the hydraulic conductivity at natural saturation (permeability in the units of inches/hour) and the storage suction factor (in inches) at field capacity, a condition comparable to the SCS AMC II. These parameters are discussed by Morel-Seytoux in Sanders, 1980. In the XSRAIN program, these parameters may be specified as input data or calculated from the SCS Curve Number by the table of correspondence.

Four main options are included in XSRAIN for the inclusion of precipitation and infiltration. They are:

1. User imposed Huff, 1967, time distribution of rainfall with field capacity soil moisture (AMC II condition) assumed

2. User imposed Huff, 1967, time distribution of rainfall with time accounting of antecedent moisture conditions

3. User specified time distribution of rainfall or balanced hyetograph with time accounting of antecedent moisture conditions

4. User specified time distribution of rainfall or balanced hyetograph with field capacity soil moisture (AMC II) assumed

Once the excess rainfall is determined, the model uses the SCS equation for lag and discretized coordinates of the SCS dimensionless unit hydrograph and mass curve (Figure 53) to derive the unit hydrograph. The flood hydrograph is then determined by multiplying the unit hydrograph by the incremental rainfall excess rate as computed by the selected option from those listed above.

## 7.0 HYDROGRAPH ROUTING

Once an appropriate design hydrograph has been prepared, it can be routed downstream and used to design or analyze a drainage structure. Two of the more common uses for routing of design hydrographs are to analyze the effects of a channel modification upon peak discharge, and to design drainage structures taking detention storage into account. Other uses for routing of design hydrographs include the design of pumping stations and the determination of the time of overtopping for highway embankments. These applications can be grouped into two categories, namely channel routing and reservoir routing. Channel routing techniques are used when the outflow from a reach of stream depends upon the inflow and storage. Reservoir routing techniques are used when outflow depends upon storage alone. These two techniques are discussed more fully in the following sections.

### 7.1 Channel Routing

Routing is a procedure by which a hydrograph at any downstream point is determined from a known hydrograph at some upstream point. As a flood hydrograph moves down a channel, its shape is modified as water is stored in the channel. The channel storage is composed of two parts: the prismatic storage which is the water in the channel when inflow and outflow are equal, and the wedge storage which is proportional to the difference between inflow and outflow. The primary characteristics of hydrograph routing are illustrated in Figure 71.

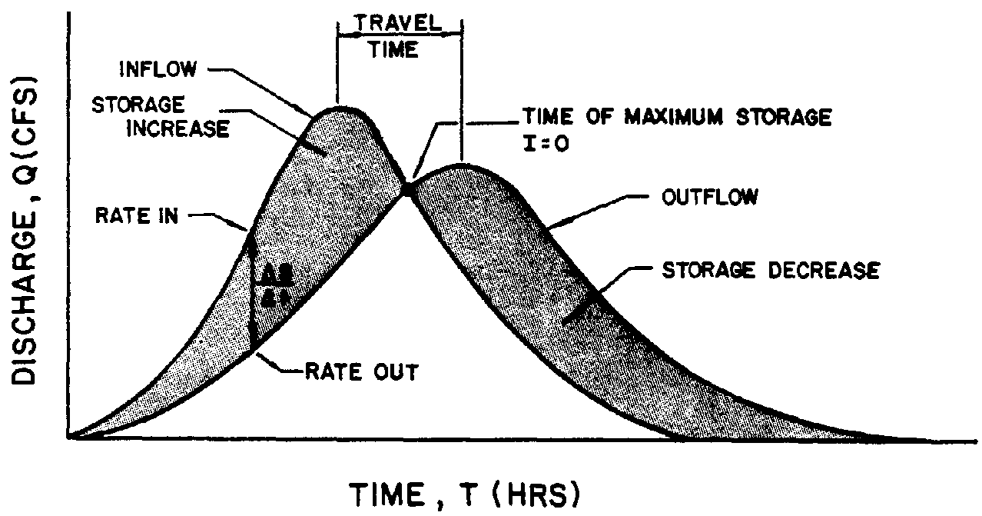



Figure 71. Inflow and Outflow Hydrographs

The general storage equation for channel routing is based on continuity and represents an accounting of all flow within a reach. Mathematically, the storage equation can be written as

$$\frac{ds}{dt} = I - O \tag{7-1}$$

where ds is the change in storage during dt in $ft^3$, dt is the change in time in sec, and I and O are the average inflow and outflow during dt, respectively, in CFS.

There are a number of techniques available for the routing of hydrographs through channels all of which are based on Equation (7-1). One of the most frequently used is the Muskingum Method which is described in this section. The Muskingum Method is based upon the assumption that the storage within a given reach of river is given by the equation below

$$s = K\left[X I + (1-X)O\right] \tag{7-2}$$

where s is the storage in $ft^3$, K is an empirical constant usually set equal to the average travel time through the reach, in consistent units, X is another empirical constant which weights the relative importance of inflow vs outflow in determining the storage (varies between 0 and 0.5), I is the inflow to the reach in CFS, and O is the outflow from the reach in CFS.

As a first step, the inflow and outflow hydrographs are divided into successive time periods, $\Delta t$, of finite duration. This duration is known as the routing period and must be smaller than the travel time through the reach so that the wave crest does not completely pass through the reach during the routing period. The differential form of the continuity equation, Equation (7-1), can be rewritten in terms of the routing period as

$$1/2(I_1 + I_2) - 1/2(O_1 + O_2) = (s_2 - s_1)/\Delta t \tag{7-3}$$

or

$$I_1 + I_2 + \frac{2s_1}{\Delta t} - O_1 = \left(\frac{2s_2}{\Delta t} + O_2\right) \tag{7-4}$$

Substituting Equation (7-2) into (7-4), the following relation is obtained.

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \tag{7-5}$$

where

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \tag{7-6}$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (7\text{-}7)$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (7\text{-}8)$$

and

$$C_0 + C_1 + C_2 = 1 \qquad (7\text{-}9)$$

and $O_2$ is the outflow at the end of $\Delta t$ in CFS, $O_1$ is the outflow at the beginning of $\Delta t$ in CFS, $I_2$ is the inflow at the end of $\Delta t$ in CFS, and $I_1$ is the inflow at the beginning of $\Delta t$ in CFS.

The application of Equation (7-5) to route an inflow hydrograph through a reach of stream is fairly straightforward. The difficulty lies in the determination of reasonable values for K and X. The preferred method is to estimate K and X using measured hydrographs; however, such data are rarely available so more approximate methods are employed.

When no other data are available, K is estimated to be the average travel time through the reach which is determined from Manning's equation. The discharge used in determining a value for K is the average discharge for the hydrograph. The value of X is estimated between 0.2 and 0.3 in the absence of any other data.

Values of K and X can also be determined from data by a trial and error process. From Equation (7-2), K can be calculated as

$$K = \frac{s}{[XI + (1-X)O]} \qquad (7\text{-}10)$$

or it is the inverse of the slope of the line of $[XI + (1-X)O]$ vs s. Values of X (between 0 and 0.5) must be assumed before the relation can be plotted. The value of X which most nearly gives a straight line is the appropriate value to use for determining K. This trial and error solution is illustrated in Figure 72 with the value K determined when $X = X_3$.

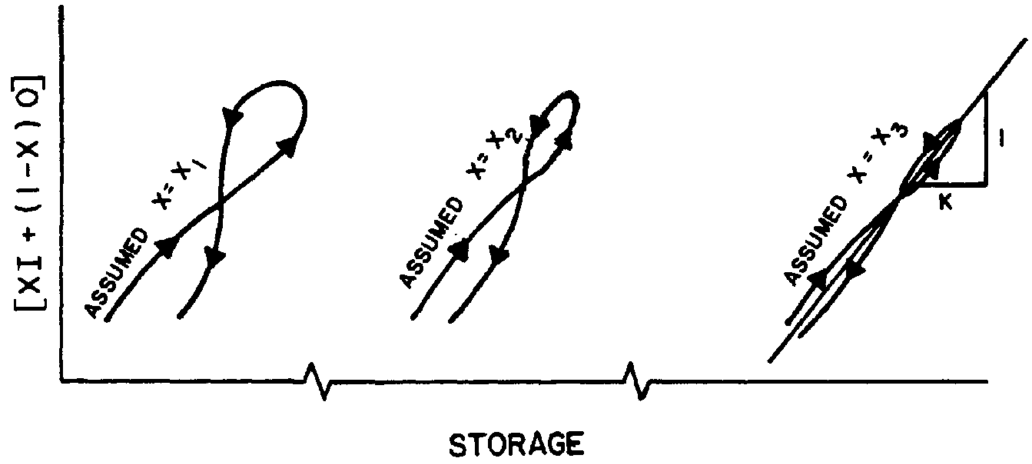
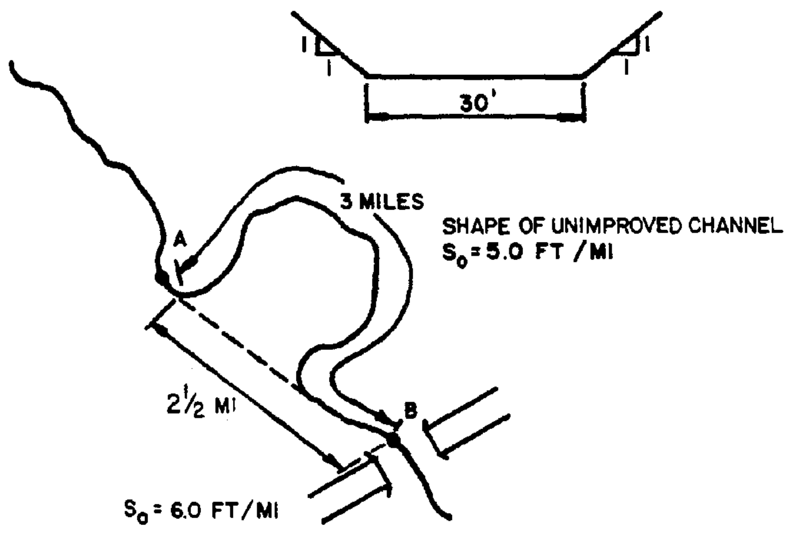



Figure 72. Valley Storage Curves

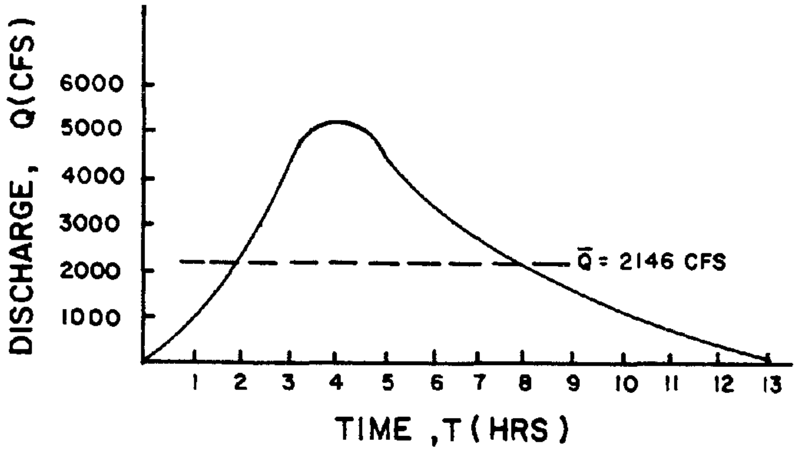
The application of the Muskingum method is illustrated by the following example:

A three mile reach of river is shown in the sketch below. A channel improvement is proposed which will cut off the meander and reduce the length of channel to 2-1/2 miles. What effect will this channel improvement have on the peak discharge experienced at the roadway at point B?




**PROPOSED CHANNEL IMPROVEMENT**

A synthetic hydrograph at Point A is developed using the procedures presented in Section 6.2 for a 25-year design discharge. The peak discharge is 5200 CFS. The design hydrograph is shown in the following sketch.




The average discharge for this hydrograph is 2146 CFS (61 CMS). Using the idealized trapezoidal cross section given in the sketch above, the average travel time is computed below

(a value of 0.025 for Manning's n is assumed)

$$V = \frac{1.49}{0.025} R^{2/3} S_o^{1/2} \quad ; \quad T = \frac{\text{LENGTH}}{\text{VELOCITY}}$$

In the unmodified 3 mile reach, the travel time is computed to be 0.70 hours. For the modified 2.5 mile reach, the travel time is computed to be 0.55 hours.

For the unmodified reach, the coefficients $C_0$, $C_1$ and $C_2$ are first computed using $\Delta t$ = 1 hour, an assumed value of X = 0.2 and K = 0.70 hours as follows

$$C_o = \frac{-0.70(0.2) + 0.5(1)}{0.70 - 0.70(0.2) + 0.5(1)} = 0.3396$$

$$C_1 = \frac{0.70(0.2) + 0.5(1)}{0.70 - 0.70(0.2) + 0.5(1)} = 0.6038$$

$$C_2 = \frac{0.70 - 0.70(0.2) - 0.5(1)}{0.70 - 0.70(0.2) + 0.5(1)} = 0.0566$$

From Equation (7-9), these values can be checked as follows

$$C_0 + C_1 + C_2 = 0.3396 + 0.6038 + 0.0566 = 1.0000$$

The outflow hydrograph ordinates can now be computed with Equation (7-5). Beginning at t = 1 hour

$$O_2 = C_0I_2 + C_1I_1 + C_2O_1 = 0.3396(800) + 0.6038(0) + 0.0566(0)$$
$$= 272 \text{ CFS } (7.7 \text{ CMS})$$

At t = 2 hours

$$O_2 = 0.3396(2000) + 0.6038(800) + 0.0566(272) = 1178 \text{ CFS } (33 \text{ CMS})$$

These values along with the remaining calculations are tabulated below.

| T (HRS) | I (CFS) | O (CFS) |
|---|---|---|
| 0 | 0 | 0 |
| 1 | 800 | 272 |
| 2 | 2000 | 1178 |
| 3 | 4200 | 2701 |
| 4 | 5200 | 4455 |
| 5 | 4400 | 4886 |
| 6 | 3200 | 4020 |
| 7 | 2500 | 3009 |
| 8 | 2000 | 2359 |
| 9 | 1500 | 1851 |
| 10 | 1000 | 1350 |
| 11 | 700 | 918 |
| 12 | 400 | 610 |
| 13 | 0 | 276 |
| 14 | 0 | 16 |
| 15 | 0 | 1 |

The same procedure is used to route the hydrograph through the modified reach. The routing coefficients are recomputed using K = 0.55, the travel time through the modified reach. The new coefficients are

$$C_0 = 0.4149$$

$$C_1 = 0.6489$$

$$C_2 = -0.0638$$

$$C_0 + C_1 + C_2 = 1.0000$$

The results of the hydrograph routing through the modified reach are summarized below

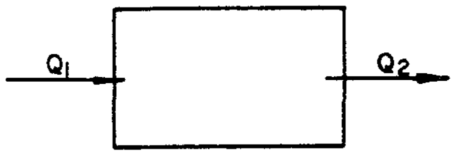
| T (HRS) | I (CFS) | O (CFS) |
|---|---|---|
| 0 | 0 | 0 |
| 1 | 800 | 332 |
| 2 | 2000 | 1328 |
| 3 | 4200 | 2956 |
| 4 | 5200 | 4694 |
| 5 | 4400 | 4900 |
| 6 | 3200 | 3870 |
| 7 | 2500 | 2867 |
| 8 | 2000 | 2269 |
| 9 | 1500 | 1775 |
| 10 | 1000 | 1275 |
| 11 | 700 | 858 |
| 12 | 400 | 565 |
| 13 | 0 | 223 |
| 14 | 0 | 0 |
| 15 | 0 | 0 |

The peak discharge at the bridge for the unmodified channel is 4886 CFS (138 CMS) and for the modified channel is 4900 CFS (139 CMS). The difference is not significant and the channel modification will have minimal effect upon the peak discharge experienced at the bridge.

## 7.2 Reservoir Routing

Whenever the outflow from a reach of river is dependent only upon the storage in the reach, the reservoir routing technique can be applied. In highway drainage design this condition is often approximated as water is backed up by a culvert and impounded (stored) by the highway embankment. Another application is in the design of detention storage basins which are often used to mitigate the increase in peak discharge associated with urbanization.

The method of reservoir routing presented in this section is the Storage-Indication method and is again based on the continuity equation.




Given the box shown above with an inflow, $Q_1$, and an outflow, $Q_2$, there is a steady-state condition as long as $Q_1$ equals $Q_2$. However, if $Q_1$ is greater than $Q_2$, the additional discharge goes into storage in the box. If $Q_2$ is greater than $Q_1$ then water stored in the box is released.

If $Q_1$ is replaced by I and $Q_2$ by O to signify the average inflow and outflow respectively, and storage is represented with the variable $\Delta s$, the relationship given as Equation (7-1) is again applicable.

$$\bar{I} - \bar{O} = \frac{\Delta s}{\Delta t} \tag{7-11}$$

This equation again can be rearranged into the form

$$I_1 + I_2 + \frac{2s_1}{\Delta t} - O_1 = \frac{2s_2}{\Delta t} + O_2 \tag{7-12}$$

This form of the equation is very useful because, if the outflow discharge, (O) is a function of storage alone then the terms on the left hand side of the equation are known and the value of $O_2$ can be determined from the terms on the right side of the equation.

To use this method requires that stage, storage, and discharge relationships be determined for the reservoir. The application of this procedure is best illustrated with an example.

Example: The designer wishes to design a culvert so that when the 50-year peak discharge is impounded the maximum water level is 1 foot below the roadway elevation. What size CMP culvert should be specified?

The hydrograph associated with the 50-year peak discharge is shown in the following table:

| Time hours | Discharge CFS |
|---|---|
| 0 | 0 |
| 1 | 20 |
| 2 | 40 |
| 3 | 60 |
| 4 | 40 |
| 5 | 20 |
| 6 | 0 |

The stage-discharge relationships for CMP culverts of various sizes are tabulated as follows

Discharge vs. Headwater Depth for Various Culvert Sizes

| Dia (ft) | Head Water Depth (ft) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 0 | 1 | 2 | 3 | 4 | 5 | 6 |
| 2.0 | 0 | 4.1 | 12.6 | 20.0 | 26.0 | 31.0 | 35.0 |
| 2.5 | 0 | 5.0 | 16.0 | 29.0 | 37.0 | 45.0 | 51.0 |
| 3.0 | 0 | 6.0 | 18.0 | 35.0 | 50.0 | 61.0 | 70.0 |
| 3.5 | 0 | 7.0 | 20.5 | 41.0 | 60.0 | 80.0 | 92.0 |
| 4.0 | 0 | 8.0 | 22.5 | 46.0 | 71.0 | 90.0 | 112.0 |

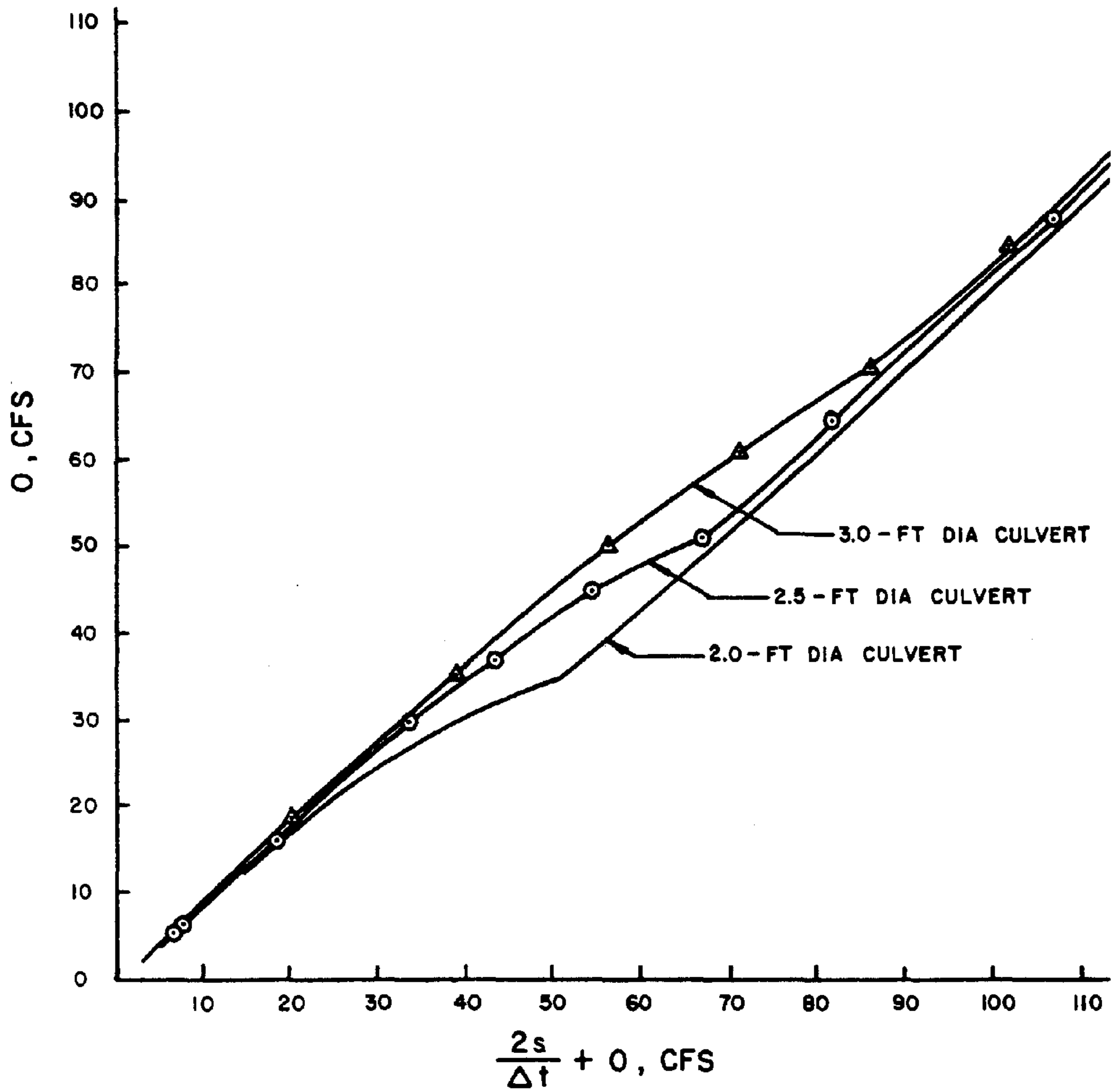
When the depth is greater than 6 feet, the embankment is overtopped and the discharge increases significantly as the embankment begins to function as a broad crested weir. At a depth of 7 feet the discharge is 170 CFS (4.8 CMS) due to overtopping alone.

The depth storage relationship is site specific. For the particular location in this example, the depth vs storage relationship is tabulated below.

| Depth (ft) | Storage ($ft^3$) | Depth (ft) | Storage ($ft^3$) |
|---|---|---|---|
| 0 | 0 | 4 | 11900 |
| 1 | 2000 | 5 | 17500 |
| 2 | 4500 | 6 | 28900 |
| 3 | 7780 | 7 | 45700 |

Using the data presented above, the values of $(\frac{2s}{\Delta t} + O)$ for the various culvert sizes are determined. Note that an appropriate value for $\Delta t$ must be selected. In this example 1 hour was chosen as convenient.

The $(\frac{2s}{\Delta t} + O)$ values determined above are then plotted vs O as follows



The following steps are then used to route the inflow hydrograph.

1. Assume an initial value for $O_1$, (usually equal to the inflow).

2. From the $(\frac{2s}{\Delta t} + O)$ vs O curve, find the value of $(\frac{2s_1}{\Delta t} + O_1)$.

3. Determine $(\frac{2s_1}{\Delta t} - O_1)$ using the equation $\frac{2s_1}{\Delta t} - O_1 = \frac{2s_1}{\Delta t} + O_1 - 2(O_1)$.

4. Determine the value of $(\frac{2s_2}{\Delta t} + O_2)$ using the equation

$$\left(\frac{2s_2}{\Delta t} + O_2\right) = I_1 + I_2 + \left(\frac{2s_1}{\Delta t} - O_1\right)$$

5. From the $(\frac{2s}{\Delta t} + O)$ curve, find the value of $O_2$ using the value of $(\frac{2s_2}{\Delta t} + O_2)$ just computed.

6. Calculate the value of $(\frac{2s_2}{\Delta t} - O_2)$ as in step 3 and continue the procedure until the hydrograph has been routed through the reservoir.

To illustrate the Storage-Indication procedure, the inflow hydrograph is first routed for the 2-foot diameter culvert in the table below

2-foot diameter culvert

| Time hrs | I CFS | $\frac{2s}{\Delta t} - 0$ CFS | $\frac{2s}{\Delta t} + 0$ CFS | 0 CFS |
|---|---|---|---|---|
| 0 | 0 | | | 0 |
| 1 | 20 | -15.7 | 24.3 | 20 |
| 2 | 40 | -20.7 | 44.3 | 32.5 |
| 3 | 60 | -40.7 | 79.3 | 60.0 |
| 4 | 40 | -24.7 | 59.3 | 42.0 |
| 5 | 20 | -20.7 | 35.3 | 28.0 |
| 6 | 0 | | -0.7 | 0.0 |
| 7 | 0 | | | |

This table shows a peak discharge of 60 CFS (1.7 CMS) which according to the stage-discharge table for CMP culverts cannot be handled by the 2-foot diameter culvert without exceeding the roadway elevation. (Recall it is desirable to keep the depth below 5 feet or 1 foot below the embankment elevation).

The same routing procedure is now applied for the 2.5- and 3-foot diameter culverts as follows:

2.5-foot diameter culvert

| Time hrs | I CFS | $\frac{2s}{\Delta t} - 0$ CFS | $\frac{2s}{\Delta t} + 0$ CFS | 0 CFS |
|---|---|---|---|---|
| 0 | 0 | | | |
| 1 | 20 | -17.0 | 23.0 | 20 |
| 2 | 40 | -31.0 | 43.0 | 37.0 |
| 3 | 60 | -36.0 | 69.0 | 52.5 |
| 4 | 40 | -34.0 | 64.0 | 49.0 |
| 5 | 20 | -19.0 | 26.0 | 22.5 |
| 6 | 0 | - 1.0 | 1.0 | 1.0 |
| 7 | 0 | | | 0 |

3-foot diameter culvert

| Time hrs | I CFS | $\frac{2s}{\Delta t} - 0$ CFS | $\frac{2s}{\Delta t} + 0$ CFS | 0 CFS |
|---|---|---|---|---|
| 0 | 0 | | | |
| 1 | 20 | -17.5 | 22.5 | 20 |
| 2 | 40 | -33.1 | 42.5 | 37.8 |
| 3 | 60 | -48.4 | 66.9 | 57.5 |
| 4 | 40 | -40.1 | 51.9 | 46.0 |
| 5 | 20 | -16.1 | 19.9 | 18.0 |
| 6 | 0 | - 3.9 | 3.9 | 3.9 |
| 7 | 0 | | | 0 |

The peak outflow discharge for the 2.5-foot culvert is 52.5 CFS (1.5 CMS) which requires a depth of slightly more than 6.0 feet. It, too, is unsatisfactory. For the 3-foot diameter culvert, a peak flow of 57.5 CFS (1.6 CMS) is obtained which can be handled with a depth less than 5 feet. A culvert diameter of 3.0 feet meets the design criteria that the maximum water level remain 1 foot below the roadway elevation.

# 8.0 URBANIZATION AND OTHER FACTORS AFFECTING PEAK DISCHARGE AND HYDROGRAPHS

Highways are relatively permanent and consequently highway drainage structures must be designed as permanent installations often with design lives of 50 years or more. As an example, 37 percent of the highway bridges on the Federal-Aid System were built before 1950. This means that almost four out of ten bridges are more than 33 years old (1983). The designer must recognize that highway drainage structures will be in place for a long time, but that the existing conditions in the drainage basin will not necessarily remain the same over that period of time. Many areas of the country have experienced significant changes in land use and tremendous urban growth.

The effects of urbanization, channelization, diversions and detention basins must be considered in the design of highway structures. Each of these factors changes the hydrologic character of a watershed, and the designer needs to be able to quantify the effects of these factors in order to assess their magnitude and, if the effects are significant, modify the design accordingly. Methods presented in the following sections provide the designer the tools needed to quantify some of these factors.

## 8.1 Urbanization

As a watershed undergoes urbanization, the peak discharge typically increases and the hydrograph becomes shorter and rises more quickly. This is due mostly to the improved hydraulic efficiency of an urbanized area. In its natural state a watershed will have developed a natural system of conveyances consisting of gullies, streams, ponds, marshes, etc., all in equilibrium with the naturally existing vegetation and physical watershed characteristics. As an area develops, typical changes made to the watershed include: 1) removal of existing vegetation and replacement with impervious pavement or buildings, 2) improvement to natural watercourses by channelization, and 3) augmentation of the natural drainage system by storm sewers and open channels. These changes tend to decrease depression storage, infiltration, detention storage and travel time. Consequently, the peak discharges increase with hydrographs becoming shorter and rising more quickly.

Two methods of quantifying the effects of urbanization are discussed in this section. The first is a procedure developed by the USGS and described by Sauer et al., 1983, for estimating flood hydrographs for ungaged watersheds. The second are the SCS methods described in TR-55, 1975.

## 8.2 U.S. Geological Survey Urban Watershed Studies

In 1978, the Federal Highway Administration contracted with the U.S. Geological Survey to conduct a nationwide survey of flood frequencies under urban conditions. The purposes of the study were to: review the literature of urban flood studies, compile a nationwide data base of flood frequency characteristics including land-use variables for urban watersheds, and define estimating techniques for ungaged urban areas. Results of the study are described in detail in USGS Water Supply Paper 2207, 1983.

A review of nearly 600 urbanized sites resulted in a final list of 269 sites which met criteria wherein at least 15 percent of the drainage area was covered with commercial, industrial or residential development; reliable flood frequency data were available for 10 or more years (either actual peak flow data or synthesized data from a calibrated rainfall-runoff model); and the period of flood frequency data was coincident with a period of relatively constant urbanization. Table 46 lists cities and metropolitan areas used in the study and is keyed with the sources of information on equivalent rural discharges for state studies listed in Appendix D. The complete data base including topographic and climatic variables, land use variables, urbanization indices and flood frequency estimates are stored in a "Statistical Analysis System" (SAS) data set accessible through the USGS National Center, Reston, VA.

The USGS study developed a procedure for quantifying the effects of urbanization on peak discharge and flood volume. Regression equations were developed which relate the peak discharge at a specified frequency to the following: 1) drainage area, 2) peak discharge for the same watershed in a rural condition and 3) a basin development factor (BDF). The basin development factor is a measure of the degree of urbanization which exists (or might exist in the future) in the watershed. The BDF is discussed in more detail in Section 8.2.2. The USGS regression equations can be used to estimate the peak discharge and corresponding hydrograph for existing conditions of urbanization, and they can also be used to estimate the peak discharge and hydrograph for future conditions. The equations for peak discharge are presented first followed by a procedure for hydrograph estimation. The urban peak flow equations are applicable to a wide variety of geographic and climatologic conditions. They can provide useful estimates of the relative impact that varying amounts of urbanization have on peak discharge and runoff. However, these estimates cannot be treated as absolutes and some judgment must be exercised in their application.

### 8.2.1 Peak Discharge Equations

Initially, the USGS study developed regression equations for urban peak flow discharge in terms of seven independent variables. Subsequently, it was found that by eliminating the less significant independent variables from the regression analyses, simpler equations could be obtained without appreciably

Table 46. Metropolitan Areas Included in Nationwide Urban Flood-Frequency Study

| State | Metropolitan area | Source of equivalent rural discharge (see references Appendix D) |
|---|---|---|
| Alabama | Birmingham | Hains(1973), Olin and Bingham(1977) |
| Arizona | Flagstaff | Roeske(1978) |
| Arizona | Tucson | Roeske (1978) |
| California | Orange County | Waananen and Crippen(1977) |
| California | Sacramento | Waananen and Crippen(1977) |
| California | San Francisco | Waananen and Crippen(1977) |
| Colorado | Boulder | Livingston(1980) |
| Colorado | Denver | Livingston(1980) |
| Connecticut | Hartford | Weiss(1975) |
| D.C. | Washington | Walker(1971), Miller(1978) |
| Delaware | Wilmington | Simmons and Carpenter(1978) |
| Georgia | Atlanta | Price(1979) |
| Hawaii | Hilo | Not Available |
| Hawaii | Honolulu | Nakahara(1980) |
| Hawaii | Kaneohe | Nakahara(1980) |
| Hawaii | Pearl City | Nakahara(1980) |
| Illinois | Chicago | Allen and Beicek(1979) |
| Illinois | Urbana | Curtis(1977) |
| Indiana | Indianapolis | Davis(1974) |
| Iowa | Iowa City | Lara(1973) |
| Kentucky | Louisville | Hannum(1976) |
| Louisiana | Baton Rouge | Neely(1976) |
| Maryland | Baltimore | Walker(1971) |
| Massachusetts | Boston | Wandle(1981) |
| Michigan | Detroit | Bent(1970) |
| Minnesota | Duluth | Guetzkow(1977) |
| Mississippi | Canton | Colson and Hudson(1976) |
| Mississippi | Hattiesburg | Colson and Hudson(1976) |
| Mississippi | Jackson | Colson and Hudson(1976) |
| Mississippi | Natchez | Colson and Hudson(1976) |
| Missouri | St. Louis | Spencer and Alexander(1978) |
| New Jersey | Newark | Stankowski(1974) |
| New Jersey | Patterson-Clif-Pass | Stankowski(1974) |
| New Jersey | Trenton | Stankowski(1974) |
| New York | Buffalo | Zembrzuski and Dunn(1979) |
| New York | New York | Zembrzuski and Dunn(1979) |
| New York | Rochester | Zembrzuski and Dunn(1979) |
| New York | Rockland County | Zembrzuski and Dunn(1979) |
| New York | Syracuse | Zembrzuski and Dunn(1979) |
| North Carolina | Charlotte | Jackson(1976) |
| North Carolina | Lenoir | Jackson(1976) |
| Ohio | Columbus | Webber and Bartlett(1976) |

Table 46. Metropolitan Areas Included in Nationwide Urban Flood-Frequency Study (continued)

| State | Metropolitan area | Source of equivalent rural discharge (see references Appendix D) |
|---|---|---|
| Oklahoma | Oklahoma City | Thomas and Corley(1977) |
| Oregon | Portland-Vancouver | Laenen(1980) |
| Pennsylvania | Harrisburg | Flippo(1977) |
| Pennsylvania | Philadelphia | Flippo(1977) |
| Pennsylvania | Pittsburgh | Flippo(1977) |
| Pennsylvania | Indiana | Flippo(1977) |
| Rhode Island | Providence | Wandle(1981) |
| Tennessee | Nashville | Randolph and Gamble(1976) |
| Texas | Austin | Schroeder and Massey(1977) |
| Texas | Dallas | Dempster(1974) |
| Texas | Ft. Worth | Dempster(1974) |
| Texas | Houston | Liscum and Massey(1980) |
| Texas | San Antonio | Schroeder and Massey(1977) |
| Washington | Portland-Vancouver | Cummans and others(1975) |
| Washington | Seattle-Tacoma | Cummans and others(1975) |

from Sauer et al. 1983

increasing the standard error of regession. Ultimately, three parameter estimating equations were developed by the USGS for peak discharges in urbanized watersheds as follows:

$$UQ_2 = 13.2\,A^{0.21}(13-BDF)^{-0.43}RQ_2^{0.73} \tag{8-1}$$

$$UQ_5 = 10.6A^{0.17}(13-BDF)^{-0.39}RQ_5^{0.79} \tag{8-2}$$

$$UQ_{10} = 9.51A^{0.16}(13-BDF)^{-0.36}RQ_{10}^{0.79} \tag{8-3}$$

$$UQ_{25} = 8.68A^{0.15}(13-BDF)^{-0.34}RQ_{25}^{0.80} \tag{8-4}$$

$$UQ_{50} = 8.04A^{0.15} (13-BDF)^{-0.32} RQ_{50}^{0.81} \tag{8-5}$$

$$UQ_{100} = 7.70A^{0.15} (13-BDF)^{-0.32} RQ_{100}^{0.82} \tag{8-6}$$

$$UQ_{500} = 7.47A^{0.16} (13-BDF)^{-0.30} RQ_{500}^{0.82} \tag{8-7}$$

where $UQ_r$ is the peak discharge of recurrence interval, r, for an urbanized condition in (CFS) where r ranges from 2 to 500 years, A is the area of the drainage basin in sq mi, BDF is the Basin Development Factor as defined below, and $RQ_r$ is the estimate of peak discharge of recurrence interval, r, for rural conditions in (CFS).

These equations are applicable for watersheds between 0.2 and 100 square miles.

### 8.2.2 Basin Development Factors

Several indices of urbanization were evaluated in the course of the USGS study but the Basin Development Factor (BDF), which provides a measure of the efficiency of the drainage system within an urbanizing watershed was selected for a number of reasons. It was highly significant in the regression equations and it is fairly easy to determine from topographic maps and field surveys. The method of determining the BDF for a watershed is explained below.

The basin is first divided into three sections as shown in Figure 73. Each section contains approximately a third of the drainage area of the watershed. Travel time is given consideration when drawing these boundaries so that the travel distances along two or more streams within a particular third are about equal. This does not mean that the travel distances of all three subareas are equal; only that within a particular subarea the travel distances are approximately equal.

Within each section of the basin, four aspects of the drainage system are evaluated and assigned a code as follows.

1. Channel improvements. If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channel and principal tributaries (those that drain directly into the main channel), then a code of one (1) is assigned. Any one, or all, of these improvements would qualify for a code of one (1). To be considered prevalent, at least 50 percent of the main drainage channel and principal tributaries must be improved to some extent over natural conditions. If channel improvements are not prevalent, then a code of zero (0) is assigned.

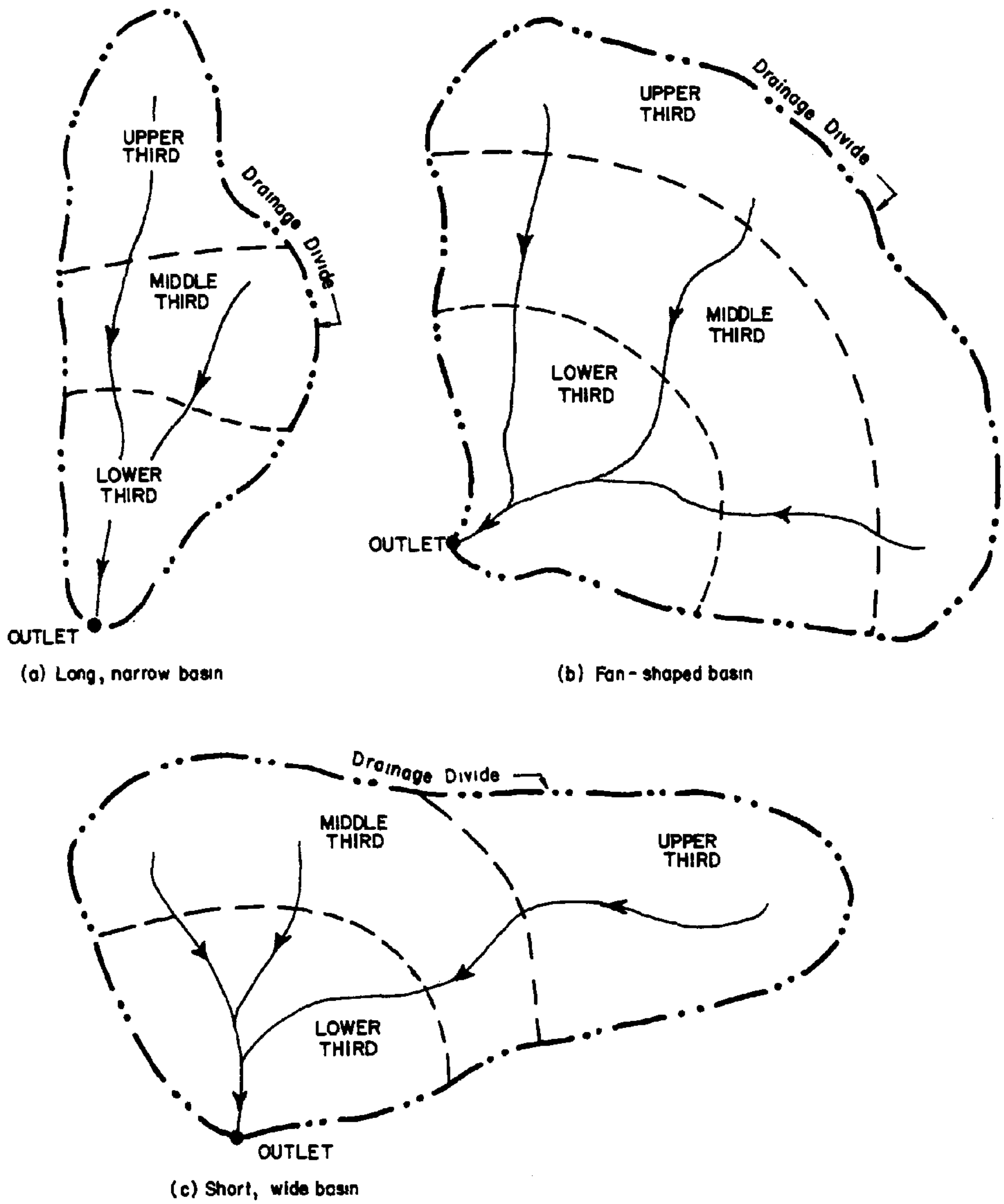



Figure 73. Subdivision of Watersheds for Determination of Basin Development Factor

2. Channel linings. If more than 50 percent of the main drainage channel and principal tributaries have been lined with an impervious material, such as concrete, then a code of one (1) is assigned. If less than 50 percent of these channels are lined, then a code of zero (0) is assigned. The presence of channel linings would probably indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.

3. Storm drains or storm sewers. Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Quite often these drains empty into the main tributaries and channel which are either open channels, or in some basins may be enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a section consists of storm drains, then a code of one (1) is assigned, and conversely if less than 50 percent of the secondary tributaries consists of storm drains, then a code of zero (0) is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, then the aspects of channel improvements and channel linings would also be assigned a code of one (1).

4. Curb and gutter streets. If more than 50 percent of a subarea is urbanized (covered by residential, commercial, and/or industrial development), and if more than 50 percent of the streets and highways in the subarea is constructed with curbs and gutters, then a code of one (1) should be assigned. Otherwise, a code of zero (0) is assigned. Frequently, drainage from curb and gutter streets will empty into storm drains.

The above guidelines for determining the various drainage system codes are not intended to be precise measurements. A certain amount of subjectivity is involved. It is recommended that field checking be performed to obtain the best estimate. The basin development factor (BDF) is computed as the sum of the assigned codes. Obviously, with three subareas per basin, and four drainage aspects to which codes are assigned in each subarea, the maximum value for a fully developed drainage system would be 12. Conversely, if the drainage system has not been developed, then a BDF of zero (0) would result. Such a condition does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, and have some improvements to secondary tributaries, and still have an assigned BDF of zero (0). It will be shown later that such a condition will still frequently cause increases in peak discharges.

The BDF is a fairly easy index to estimate for an existing urban basin. The 50 percent guideline is usually not difficult to evaluate because many urban areas tend to use the same design criteria throughout, and therefore the drainage aspects are similar throughout. Also, the BDF is convenient to use for projecting future development. Obviously, full development and maximum urban effects on peaks would occur when BDF = 12. Projections of full development, or intermediate stages of development, can usually be obtained from city engineers.

Example: BDF Calculation

The following summary represents information collected from topographic maps and a field survey on a given watershed. Determine the BDF for the drainage basin given the following data:

Total Length of Main Channel: 100 miles

Total Length of Secondary Tributaries:
- Upper Third: 160 miles
- Middle Third: 100 miles
- Lower Third: 80 miles

Total Road Miles:
- Upper Third: 100 miles
- Middle Third: 140 miles
- Lower Third: 200 miles

| | | |
|---|---|---|
| **Channel Improvements** | | |
| Upper Third: | 22 miles have been straightened & deepened. | Code = 1 |
| Middle Third: | 10 miles have been straightened & deepened. | = 0 |
| Lower Third: | 27 miles have been straightened & widened. | = 1 |
| **Channel Linings** | | |
| Upper Third: | 6 miles of channel are lined. | Code = 0 |
| Middle Third: | 10 miles of channel are lined. | = 0 |
| Lower Third: | 24 miles of channel are lined. | = 1 |
| **Storm Drains on Secondary Tributaries** | | |
| Upper Third: | 40 miles have been converted to drains. | Code = 0 |
| Middle Third: | 72 miles have been converted to drains. | = 1 |
| Lower Third: | 68 miles have been converted to drains. | = 1 |
| **Curb and Gutter Streets** | | |
| Upper Third: | 20 miles | Code = 0 |
| Middle Third: | 90 miles | = 1 |
| Lower Third: | 150 miles | = 1 |
| | | BDF = 7 |

Example: What is the 25-year peak discharge for an urban watershed of 26 square miles with a BDF of 4? What is the percentage increase over the equivalent rural watershed?

1. Determine the equivalent rural discharge using the published USGS statewide regression equations. For this site the 25-year peak discharge for the rural conditions is determined from the following equation:

$$RQ_{25} = 280\ A^{0.666}$$

$$RQ_{25} = 280\,(26)^{0.666} = 2450 \text{ CFS } (69 \text{ CMS})$$

2. Determine the urban discharge.

$$UQ_{25} = 8.68\ A^{0.15}(13 - BDF)^{-0.34} RQ_{25}^{0.80}$$

$$UQ_{25} = 8.68(26)^{0.15}(13 - 4)^{-0.34}(2450)^{0.80} = 3450 \text{ CFS } (98 \text{ CMS})$$

The 25-year peak discharge for the urban watershed is 3450 CFS (98 CMS).

3. Determine the percent change.

$$\frac{UQ_{25} - RQ_{25}}{RQ_{25}} \times 100$$

$$\frac{3450 - 2450}{2450} \times 100 = 41\%$$

The regression equations can also be used to determine the effects of future urbanization upon peak discharges. This calculation is simplified by performing some algebraic manipulation of the regression equations.

Example: What percentage increase in the 5-year peak discharge results when the BDF changes from 5 to 10?

The present $\quad UQ_5 = 10.6\ A^{0.17}(13 - BDF_p)^{-0.39} RQ^{0.78}$

where: $BDF_p$ = the present BDF

The future $\quad UQ_5 = 10.6\ A^{0.17}(13 - BDF_f)^{-0.39} RQ^{0.78}$

where $BDF_f$ = the future BDF

Letting $\quad \Delta BDF = (BDF_f - BDF_p)$

then $\quad BDF_f = BDF_p + \Delta BDF$

The ratio of the future $UQ_5$ to the present $UQ_5$ is

$$\frac{UQ_{5F}}{UQ_{5P}} = \frac{\cancel{10.6A^{0.17}}\,[13-(BDF_p+\Delta BDF)]^{-0.39}\,\cancel{RQ^{0.78}}}{\cancel{10.6A^{0.17}}\,(13-BDF_p)^{-0.39}\,\cancel{RQ^{0.78}}}$$

Cancelling the common terms and rearranging yields

$$\frac{UQ_{5F}}{UQ_{5P}} = \left[1 - \frac{\Delta BDF}{13-BDF_p}\right]^{-0.39}$$

for the example at hand, $BDF_p = 5$ and $\Delta BDF = (10 - 5)$

Therefore

$$\frac{UQ_{5F}}{UQ_{5P}} = \left[1 - \frac{5}{8}\right]^{-0.39} = 1.47$$

The future 5-year peak discharge is 47 percent higher than the present 5-year peak discharge.

The same approach can be applied to the other recurrence intervals yielding the following general equation

$$\frac{UQ_F}{UQ_P} = \left[1 - \frac{\Delta BDF}{(13-BDF_P)}\right]^n \qquad (8\text{-}8)$$

where n varies with recurrence intervals as given in Table 47.

Table 47. Variation of BDF Exponent with Recurrence Interval

| $T_r$ | n |
|---|---|
| 2 | -0.43 |
| 5 | -0.39 |
| 10 | -0.36 |
| 25 | -0.34 |
| 50 | -0.32 |
| 100 | -0.32 |

### 8.2.3 Hydrograph Equation

Using the regression equations presented above, it is possible to determine a peak discharge for an urbanizing watershed for a number of recurrence intervals. If a corresponding hydrograph is needed for these peak discharges the procedure presented below can be used. This method was developed by the USGS based upon a study of 62 stations in various geographic locations for which calibrated rainfall-runoff models existed. These stations are a subset of the 269 gaged basins used to develop the previous peak discharge equations. The results are applicable to a wide range of geographic and climatic conditions. The resulting hydrograph should be as accurate as other synthetic hydrographs.

A standardized dimensionless hydrograph was developed by Stricker and Sauer, 1982, which is used for all watersheds. The ordinates of the hydrograph are given in terms of their ratio to the estimated peak discharge. The time scale of the hydrograph is given in terms of its ratio to the basin lag time. The dimensionless hydrograph is shown in Figure 74 and its ordinates are tabulated in Table 48.

Table 48. Time and Discharge Ratios of the Dimensionless Urban Hydrograph

| Time ratio ($t/T_L$) | Discharge ratio ($Q_t/Q_p$) |
|---|---|
| .45 | .27 |
| .50 | .37 |
| .55 | .46 |
| .60 | .56 |
| .65 | .67 |
| .70 | .76 |
| .75 | .86 |
| .80 | .92 |
| .85 | .97 |
| .90 | 1.00 |
| .95 | 1.00 |
| 1.00 | .98 |
| 1.05 | .95 |
| 1.10 | .90 |
| 1.15 | .84 |
| 1.20 | .78 |
| 1.25 | .71 |
| 1.30 | .65 |
| 1.35 | .59 |
| 1.40 | .54 |
| 1.45 | .48 |
| 1.50 | .44 |
| 1.55 | .39 |
| 1.60 | .36 |
| 1.65 | .32 |
| 1.70 | .30 |

from Stricker and Sauer, 1982

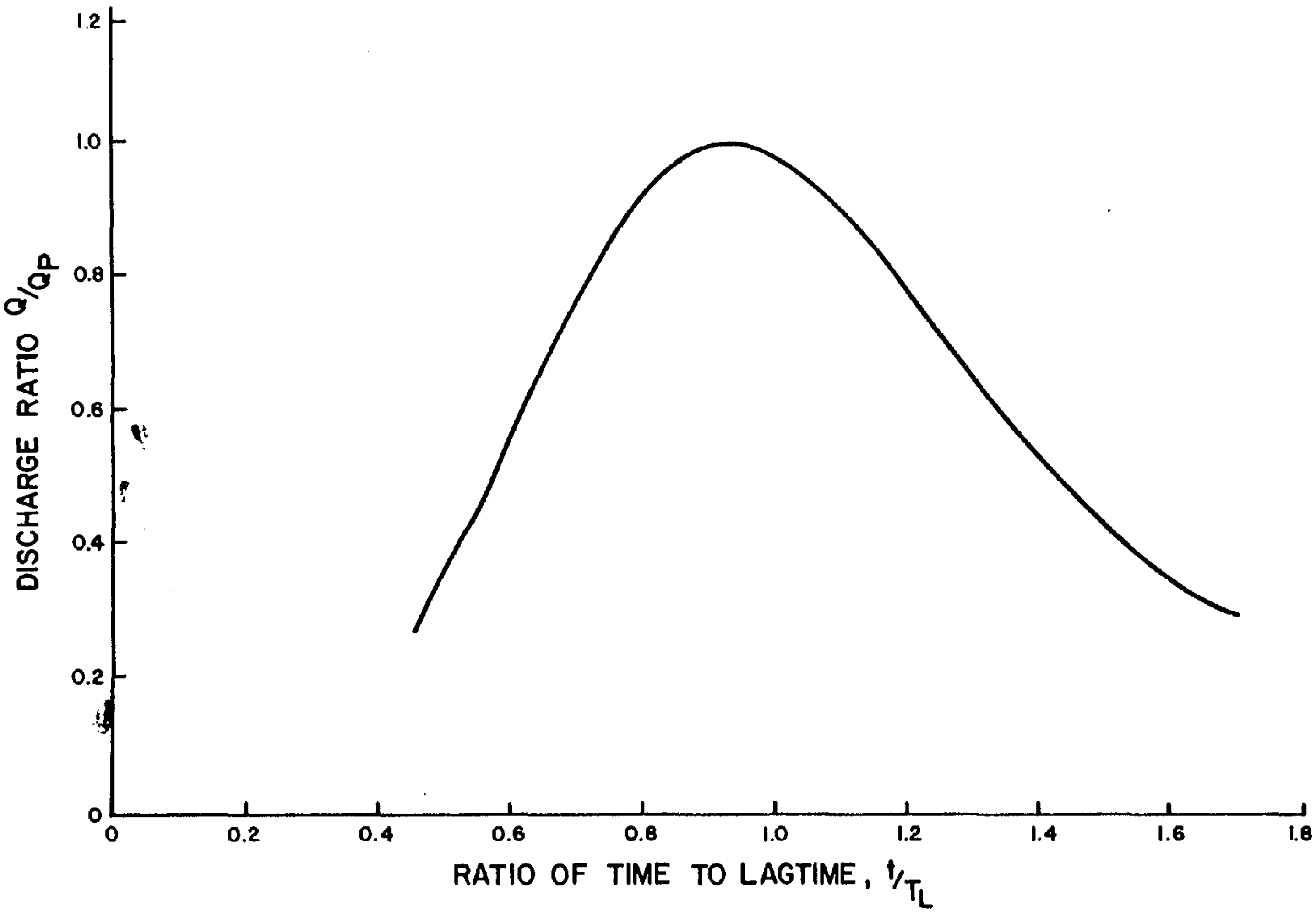



Figure 74. Dimensionless USGS Urban Hydrograph

To develop this hydrograph, an estimate of the basin lag time is necessary. The USGS developed the following equation for estimating basin lag time

$$T_L = 0.85\ L^{0.62}\ ST^{0.31}\ (13 - BDF)^{0.47} \qquad (8\text{-}9)$$

where $T_L$ is the lag time for the urban watershed in hrs, L is the basin length from the outlet to the watershed divide in mi, ST is the main channel slope in ft/mi, measured between points which are 10 and 85 percent of the main channel length, and BDF is the basin development factor as defined in the previous section. (ST is not to be greater than 70 ft/mi. If ST is greater than 70 ft/mi, use 70 ft/mi).

Using Equation (8-9) and the peak discharge equations presented in the previous section, it is possible to construct a hydrograph in accordance with the following stepwise procedure.

1. From the best available topographic maps, determine the drainage area, main-channel length, and main-channel slope of the basin.

2. Compute the equivalent rural peak discharge from the applicable U.S. Geological Survey flood-frequency reports (Appendix D).

3. Compute the basin development factor. This parameter can be easily determined using drainage maps and by making field inspections of the drainage basin.

4. Compute the urban peak discharge using the appropriate equation for the selected frequencies given in Section 8.2.1.

5. Compute the lagtime from Equation (8-9).

6. For some situations an entire hydrograph may not be needed. An estimate of the width of the hydrograph for a specific discharge, Q, may be enough to estimate the time that flow will inundate a specific structure, such as a road embankment. This time, $t_w$, can be obtained by calculating the ratio $Q/Q_p$. Using $Q/Q_p$ to determine a value of $t_w/T_L$ from Figure 74, and multiplying the lagtime, $T_L$, by the ratio $t_w/T_L$, will give the hydrograph width or time that flow is greater than the specified Q. The recurrence interval corresponds to the recurrence interval of $Q_p$.

7. The coordinates of the runoff hydrograph can be computed by multiplying the value of lag time by the time ratios and the value of peak discharge by the discharge ratios presented in Table 48.

Example

The procedure is illustrated in an example taken from Jackson, 1976, to compute a hydrograph associated with the 100-year discharge estimated for Little Sugar Creek at Charlotte, N.C.

1. The drainage area (A) is determined as 41 sq mi and the basin length (L) and slope (ST) are determined to be 11 mi and 13.1 ft/mi, respectively.

2. The equivalent rural peak discharge ($RQ_{100}$) for the 100-year recurrence-interval flood is 7,460 CFS (211 CMS), Jackson, (1976).

3. The basin development factor (BDF) is computed to be 9.

4. Using Equation (8-6), the urban peak discharge for the 100-year recurrence-interval flood ($UQ_{100}$) is estimated to be

$$
\begin{aligned}
UQ_{100} &= 7.70A^{0.15}(13-BDF)^{-0.32}RQ_{100}^{0.82} \\
&= (7.70)(41)^{0.15}(13-9)^{-0.32}(7460)^{0.82} \\
&= 12{,}900 \text{ CFS } (365 \text{ CMS})
\end{aligned}
$$

5. Using Equation (8-9), lagtime ($T_L$) is estimated to be

$$
\begin{aligned}
T_L &= 0.85\,(L)^{0.62}(ST)^{-0.31}(13-BDF)^{0.47} \\
&= (0.85)(11)^{0.62}(13.1)^{-0.31}(13-9)^{0.47} \\
&= 3.2 \text{ HRS}
\end{aligned}
$$

6. The hydrograph is computed from the dimensionless ratios in Table 48 as shown below. The resulting hydrograph is plotted in Figure 75.

7. If an estimate were needed for a time of road overtopping at a discharge of 9,000 CFS (255 CMS), it is computed as follows

   a. $Q/Q_p = 9000 / 12{,}900 = 0.70$

   b. from Table 48,

      beginning of overtopping: $(t/T_L)_b = 0.667$

      end of overtopping: $(t/T_L)_e = 1.263$

   c. lagtime

d. road overtopping time, W

$$W = \left[(t/T_L)_e - (t/T_L)_b\right] T_L$$

$$W = \left[1.263 - 0.667\right](3.2) = 1.9 \text{ HRS}$$

e. The time of overtopping can also be obtained from the hydrograph as shown in Figure 75.

| $t/T_L$ | time(hr) (3.2 X col. 1) | $Q_t/Q_p$ | Discharge(CFS) (12,900 X col.3) |
|---|---|---|---|
| .45 | 1.4 | .27 | 3,500 |
| .50 | 1.6 | .37 | 4,800 |
| .55 | 1.8 | .46 | 5,900 |
| .60 | 1.9 | .56 | 7,200 |
| .65 | 2.1 | .67 | 8,600 |
| .70 | 2.2 | .76 | 9,800 |
| .75 | 2.4 | .86 | 11,100 |
| .80 | 2.6 | .92 | 11,900 |
| .85 | 2.7 | .97 | 12,500 |
| .90 | 2.9 | 1.00 | 12,900 |
| .95 | 3.0 | 1.00 | 12,900 |
| 1.00 | 3.2 | .98 | 12,600 |
| 1.05 | 3.4 | .95 | 12,200 |
| 1.10 | 3.5 | .90 | 11,600 |
| 1.15 | 3.7 | .84 | 10,800 |
| 1.20 | 3.8 | .78 | 10,100 |
| 1.25 | 4.0 | .71 | 9,200 |
| 1.30 | 4.2 | .65 | 8,400 |
| 1.35 | 4.3 | .59 | 7,600 |
| 1.40 | 4.5 | .54 | 7,000 |
| 1.45 | 4.6 | .48 | 6,200 |
| 1.50 | 4.8 | .44 | 5,700 |
| 1.55 | 5.0 | .39 | 5,000 |
| 1.60 | 5.1 | .36 | 4,600 |
| 1.65 | 5.3 | .32 | 4,100 |
| 1.70 | 5.4 | .30 | 3,900 |

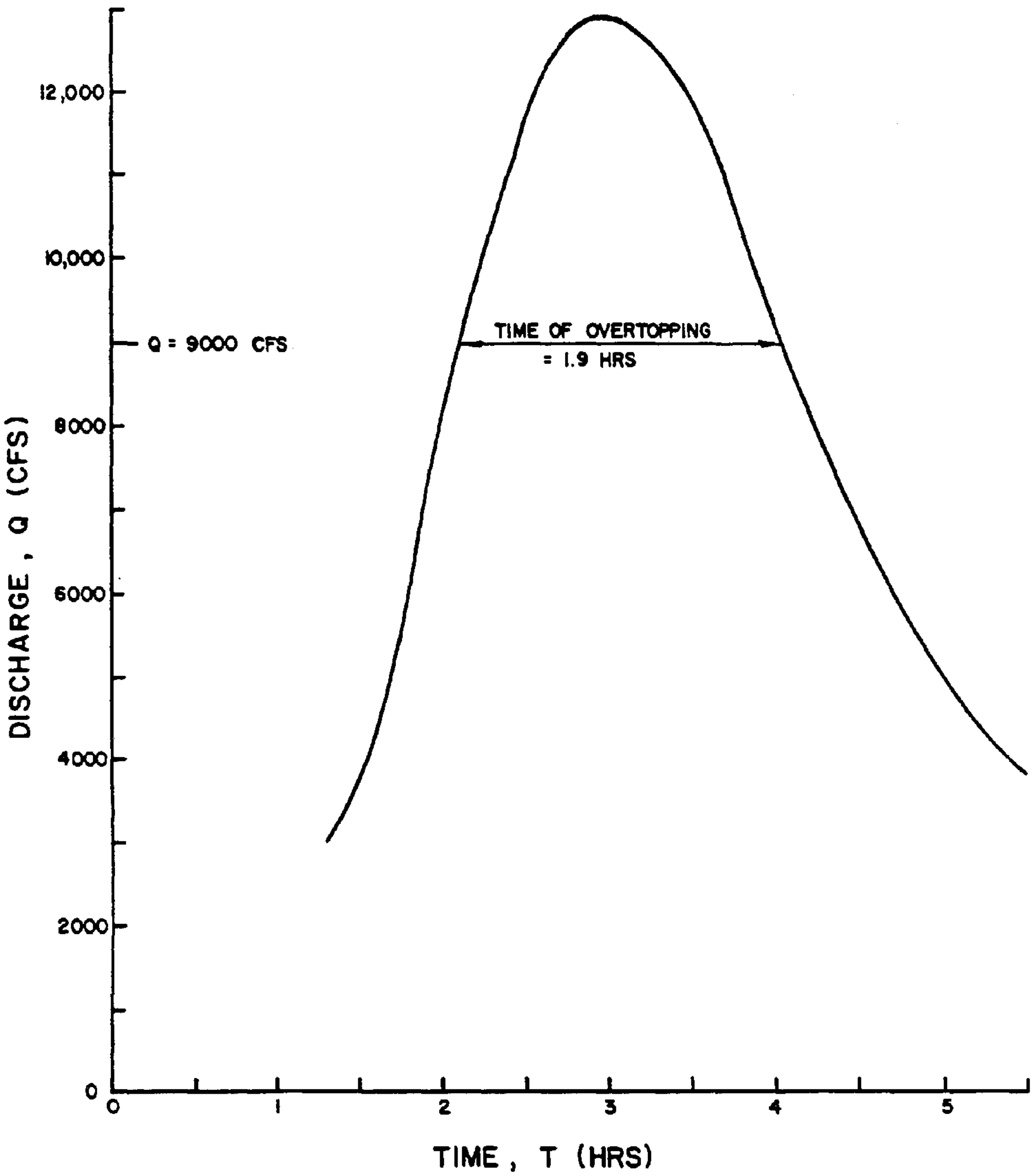



Figure 75. Urban Hydrograph for Little Sugar Creek, N.C., USGS Dimensionless Hydrograph Method

## 8.3 Soil Conservation Service TR-55 Urban Hydrology Procedures

The Soil Conservation Service has published Technical Release No. 55 (TR-55), 1975, which details procedures for quantifying the effects of urbanization upon the peak discharge and runoff hydrograph for small urban watersheds.

TR-55 describes two general methods for estimating peak discharges from urban watersheds.

1. the Graphical Method
2. the Tabular Method

The graphical method, discussed in Section 8.3.4, uses the time of concentration ($T_c$) for an urban drainage area from which the peak discharge per unit area per inch of direct runoff is obtained. This method is limited to small watersheds in which the runoff characteristics are fairly uniform and the land use, soils and ground cover can be represented by a single Curve Number (CN). The graphical method provides only a peak discharge estimate and therefore is applicable to those design situations where a hydrograph is not required.

The tabular method, Section 8.3.5, is a more complete approach and can be used to develop a composite hydrograph at any point within a watershed. The drainage area is divided into subareas with uniform runoff characteristics and a hydrograph is developed for each subbasin based on its respective Curve Number. The hydrographs are then routed through the watershed and combined to produce the composite hydrograph at the point of interest. Because of the hydrograph routing, the tabular method requires an estimate of travel time ($T_t$) in addition to the time of concentration. The tabular method is particularly useful to evaluate the effects of changed land use in a part of the watershed. It can also be used to determine the effects of structures or combinations of structures including channel modifications at different locations in an urban watershed.

Prior to using either the graphical or tabular methods, the designer must determine present and future (urban) values of the Curve Number (CN), the time of concentration ($T_c$) and the volume of runoff from a given depth of precipitation. Methods for determining these values under present or "as is" conditions were discussed in Section 6.3. The next two subsections of this manual discuss the adjustments of these parameters to account for urban effects, primarily the encroachment of impervious cover and channel improvements.

The reader is strongly encouraged to obtain a copy of TR-55 from the Soil Conservation Service. The addresses of the local offices are included in Appendix C. The analytical procedure is summarized here and an example problem is presented.

### 8.3.1 Composite Curve Number

The procedure presented in TR-55 is based upon the soil-cover-complex method discussed in Section 6.2.2.3. The effect of hydrologic soil-cover complex on runoff is expressed in terms of a runoff curve number, CN. This runoff curve number varies with land use and hydrologic soil group. Values for typical urban land uses are tabulated in Table 49. If the land use for a watershed is varied, a weighted CN can be computed based upon the relative areas. The use of weighted CN values was discussed in Section 6.2.2.3 and is further illustrated in the following example of an urbanized watershed.

Example: For a 1000 acre watershed, the hydrologic soil group is classified as B group with the following land use pattern

| Land Use | Percent |
|---|---|
| Detached houses with 1/4 acre lots | 50 |
| Townhouses with 1/8 acre lots | 10 |
| Streets with curb, plazas, etc. | 25 |
| Open space, parks, etc. | 15 |
| | 100 |

The weighted curve number is computed as shown below using Table 49.

| Land Use | Percent | CN | Product |
|---|---|---|---|
| Detached houses | 50 | 75 | 3750 |
| Town houses | 10 | 85 | 850 |
| Streets | 25 | 98 | 2450 |
| Open Spaces | 15 | 61 | 915 |
| | | | 7965 |

$$\text{Weighted CN} = \frac{7965}{100} = 80$$

The curve numbers in Table 49 are based upon average percentages of imperviousness. If the percent impervious is different from that assumed in Table 49 then the values derived in Figure 76 can be used to correct CN for other percentages of impervious cover.

Table 49. Runoff Curve Numbers for Selected Agricultural, Suburban and Urban Land Use. (Antecedent Moisture Condition II and Ia = 0.25)

| LAND USE DESCRIPTION | | HYDROLOGIC SOIL GROUP A | B | C | D |
|---|---|---|---|---|---|
| Cultivated land: | without conservation treatment | 72 | 81 | 88 | 91 |
| | with conservation treatment | 62 | 71 | 78 | 71 |
| Pasture or range land: | poor condition | 68 | 79 | 86 | 89 |
| | good condition | 39 | 61 | 74 | 80 |
| Meadow: | good condition | 30 | 58 | 71 | 78 |
| Wood or Forest land: | thin stand, poor cover, no mulch | 45 | 66 | 77 | 83 |
| | good cover | 25 | 55 | 70 | 77 |
| Open Spaces, lawns, parks, golf courses, cemeteries, etc | | | | | |
| good condition: | grass cover on 75% or more of the area | 39 | 61 | 74 | 80 |
| fair condition: | grass cover on 50% to 75% of the area | 49 | 69 | 79 | 84 |
| Commercial and business areas (85% impervious) | | 89 | 92 | 94 | 95 |
| Industrial districts (72% impervious). | | 81 | 88 | 91 | 93 |
| Residential | | | | | |
| Average lot size | Average % Impervious | | | | |
| 1/8 acre or less | 65 | 77 | 85 | 90 | 92 |
| 1/4 acre | 38 | 61 | 75 | 83 | 87 |
| 1/3 acre | 30 | 57 | 72 | 81 | 86 |
| 1/2 acre | 25 | 54 | 70 | 80 | 85 |
| 1 acre | 20 | 51 | 68 | 79 | 84 |
| Paved parking lots, roofs, driveways, etc. | | 98 | 98 | 98 | 98 |
| Streets and roads: | | | | | |
| paved with curbs and storm sewers | | 98 | 98 | 98 | 98 |
| gravel | | 76 | 85 | 89 | 91 |
| dirt | | 72 | 82 | 87 | 89 |

from SCS, 1975

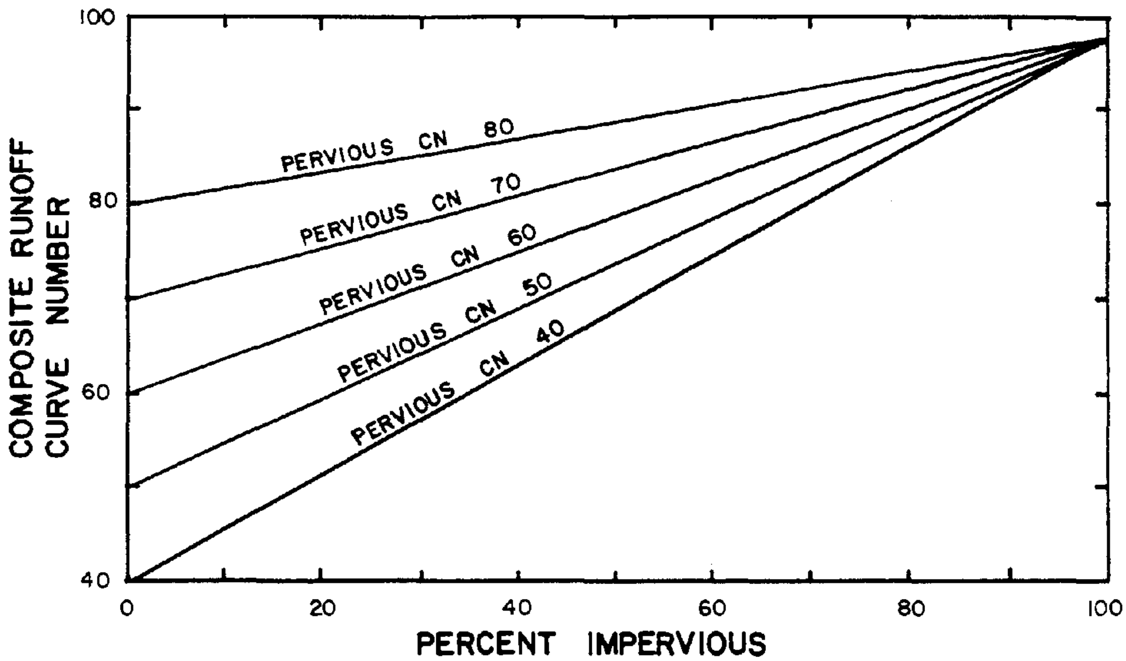



Figure 76. Composite Curve Numbers as a Function of Impervious Cover and Pervious CN Values

To demonstrate the use of Figure 76, consider the following example.

What is the weighted Curve Number for a 1000 acre watershed with hydrologic soil group C? Forty percent of the watershed is impervious, sixty percent is pervious and considered to be in good grass cover.

1. From Table 49, the pervious CN = 79
2. From Figure 76, the composite value of CN = 85

Once a weighted CN has been determined for a watershed, the volume of runoff resulting from a given depth of precipitation is found by solving the following equation

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \text{Direct runoff in inches} \qquad (8\text{-}10)$$

where $$S = \frac{1000}{CN} - 10$$

and P is the total depth of precipitation in inches, Q is the direct runoff in inches, S is the potential abstraction in inches, and CN is the weighted curve number.

Equation (8-10) is the basic equation from which Table 41 and Figures 56a and 56b are derived (Sec. 6.3).

Example: For P = 6.0 inches and CN = 84, find Q.

$$S = \frac{1000}{84} - 10 = 1.90$$

$$Q = \frac{(6 - 0.2(1.9))^2}{(6 + 0.8(1.9))} = 4.2 \text{ inches}$$

Urbanization also affects the time of concentration in the watershed. Time of concentration is the total time for water to travel from the most hydraulically remote point on the watershed to the point of interest (usually the watershed outlet). The SCS presents two methods to adjust for the effect of urbanization on time of concentration, namely

1. Modified Curve Number Method

2. Total Travel Time Method

8.3.2 Modified Curve Number Method for Time of Concentration

This is an approximate method for quantifying the effects of urbanization on the time of concentration by using the future condition curve number. The future condition time of concentration is determined using the methods of Section 6.2.2. This value is then adjusted using the following equation

$$T_{CF} = T_{CF'}[CF][IF] \tag{8-11}$$

where $T_{CF}$ is the time of concentration for future conditions in hrs, $T_{CF}'$ is the time of concentration for future conditions without channel and impervious factors considered in hrs, [CF] is the channel improvement factor defined below, and [IF] is the impervious factor defined below.

8.3.2.1 Channel Improvement Factor

Equation (8-11) is based on observations of a number of small urban watersheds and is not sufficiently refined to evaluate specific types of improvements. The channel improvement factor [CF] is found from Figure 77 and is a function of the future curve number and the percent of the main channel which has been hydraulically modified. This includes all types of modifications from straightening and lining to bank protection.

Figure 77 applies to watersheds where the natural condition of the main channel has been hydraulically improved. If the main channel has not been modified, the lag computed by Equation (8-12) can be used

$$T_L = \frac{L^{0.8} (S + 1)^{0.7}}{1900 \ LS^{0.5}} \tag{8-12}$$

where $T_L$ is the lag time in hrs, L is the hydraulic length of the watershed in ft, and LS is the average watershed land slope in percent.

The channel improvement factor [CF] is then found from Figure 77.

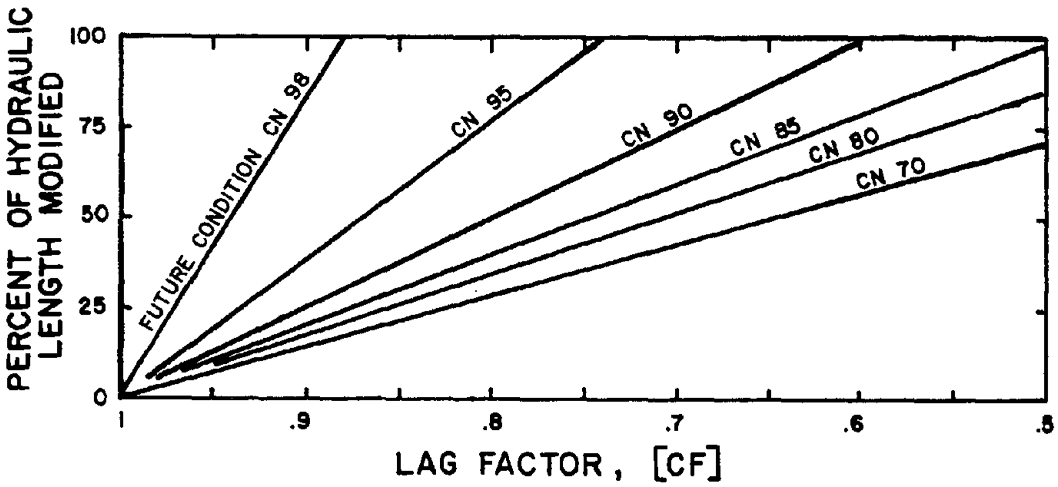



Figure 77. Factors for Adjusting Lag When the Main Channel has Been Hydraulically Improved

Not enough data are available, nor is there an equation accurate enough to distinguish between the types of channel modification made. The adjustment for channel improvement is made as follows. If 50 percent of the channel has been modified from its natural condition and the future-condition curve number is computed to be 80, then the channel improvement factor is 0.7.

#### 8.3.2.2 Impervious Factor

Figure 78 shows the impervious factor for adjusting Equation (8-11) if part of the watershed is impervious. If the future-condition curve number is 100 or the impervious area is zero, adjustments are not necessary. When a significant part of the watershed is impervious, time of concentration is decreased because the flow paths to the main channel are more efficient than under natural conditions.

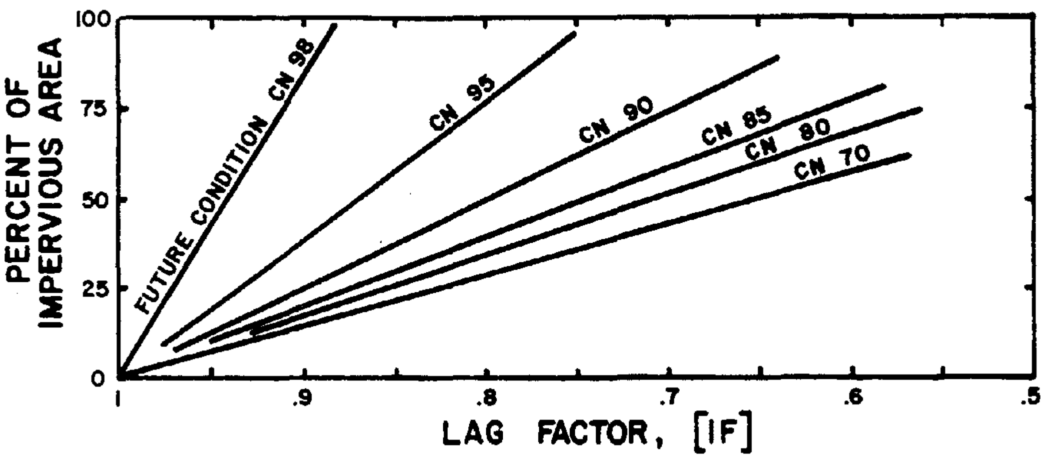



Figure 78. Factors for Adjusting Lag When Impervious Areas Occur in Watershed

Since the figures above are used only with future-condition curve numbers, the factors cannot be used directly to compute the decrease in time of concentration from present conditions. To determine the change in time of concentration from present to future conditions, it is first necessary to compute the present time of concentration and then using the future-condition curve number, compute the corresponding future value.

Example: Modified Curve Number Method taken from TR-55, SCS, 1975

A watershed of 1,000 acres has a present-condition curve number of 75, average watershed slope of 4 percent, and hydraulic length of 13,200 feet. Urban development is expected to modify about 70 percent of the hydraulic length, increase the impervious area to 40 percent, and increase the runoff curve number to 80. Compute the future condition time of concentration using the curve number method.

1. Future-condition time of concentration from Equation (8-12)

   a. Basin future-condition lag with CN = 80

$$T_L = \frac{(13200)^{0.8}(2.5+1)^{0.7}}{1900\,(4)^{0.5}} = 1.25 \text{ HRS}$$

and from Equation (6-11)

$$T_{CF'} = 1.67\,(1.25) = 2.09 \text{ HRS}$$

2. Channel improvement factor for modification of 70 percent of the hydraulic length is read from Figure 77.

$$[CF] = 0.59$$

3. The impervious factor is determined from Figure 78 for an impervious area of 40 percent.

$$[IF] = 0.77$$

4. The time of concentration for future conditions with channel improvements and impervious cover is then

$$T_{CF} = T_{CF'} [CF][IF] = 2.09 [0.59][0.77] = 0.95 \text{ HRS}$$

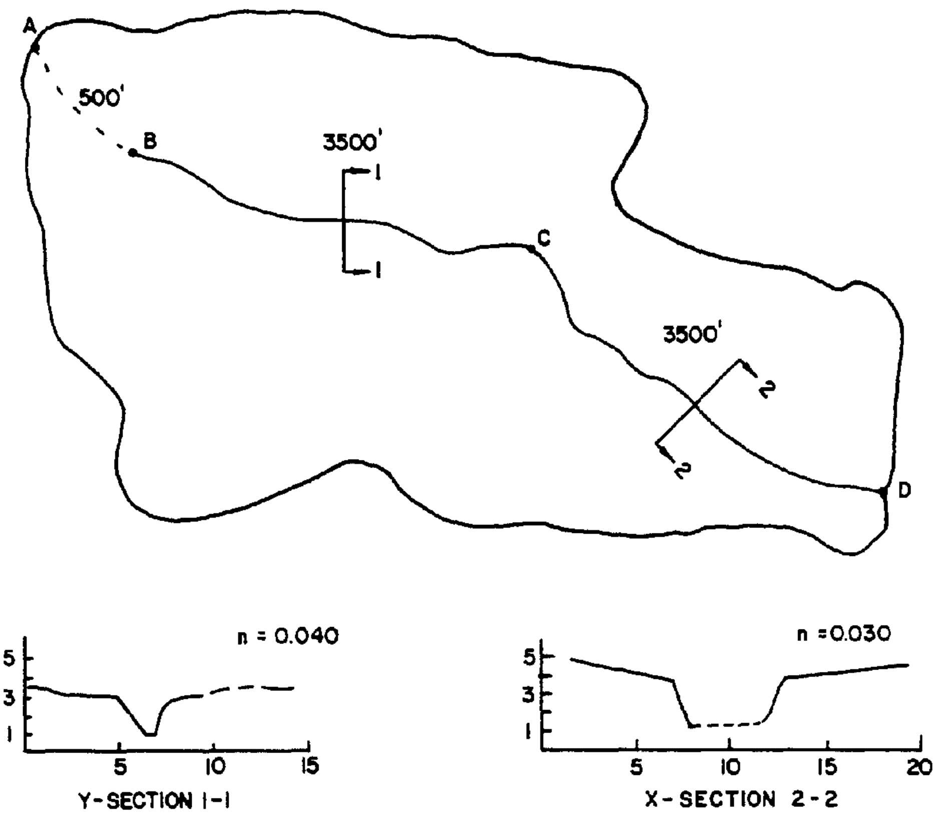
8.3.3 Total Travel Time Method for Time of Concentration

In this method the time of concentration is determined by estimating the contribution for each phase of flow (i.e., overland, storm sewer and gutter and channel flow) for present conditions and then again for future conditions. The methods used are the same as those presented in Secs. 6.2.2.1 and 6.2.2.2 for the SCS Synthetic Unit Hydrograph procedure. This method has the advantage of allowing specific changes to be quantified but requires more data than the curve number method presented above.

Example: Total Travel Time Method from TR-55, SCS, 1975

The present conditions of a small watershed are illustrated in the sketch below and summarized as follows

| Reach | Description of Flow | Slope Percent | Length |
|---|---|---|---|
| A to B | Overland (forest) | 7 | 500' |
| B to C | Natural Channel (X-Section 1-1) | 1.2 | 3500' |
| C to D | Natural Channel (X-Section 2-2) | 0.6 | 3500' |




<u>For the Present Condition</u>

1. Compute the overland flow travel time:
   Reach A to B (forest cover) from Figure 52 for a slope of 7 percent, V = 0.7 ft/sec.

$$T_t = \frac{500 \text{ ft}}{0.7 \text{ ft/sec}} = 714 \text{ sec}$$

2. Compute the natural channel travel time:
   Reach B to C the natural channel is approximated with a trapezoidal channel with (b = 1, d = 2, z = 2:1 n = 0.040).

   Using Manning's equation and computing bank full velocity

$$V = \frac{1.49}{n} R_h^{2/3} S^{1/2} \quad R_h = 1.005' \quad S = 0.012$$

$$V = \frac{1.49}{0.040} (1.005)^{2/3} (0.012)^{1/2} = 4.1 \text{ ft/sec}$$

$$T_t = \frac{3500 \text{ ft}}{4.1 \text{ ft/sec}} = 854 \text{ sec}$$

3. Compute the natural channel travel time: Reach C to D, Trapezoidal Channel (b = 4, d = 2, z = 2:1, n = 0.030)

Again using Manning's Equation

$$V = \frac{1.49}{n} \quad R_n^{2/3} \; S^{1/2} \quad R_n = 1.24' \quad S = 0.006$$

$$V = \frac{1.49}{0.030} \; (1.24)^{2/3} \; (0.006)^{1/2} \; = 4.4 \text{ ft /sec}$$

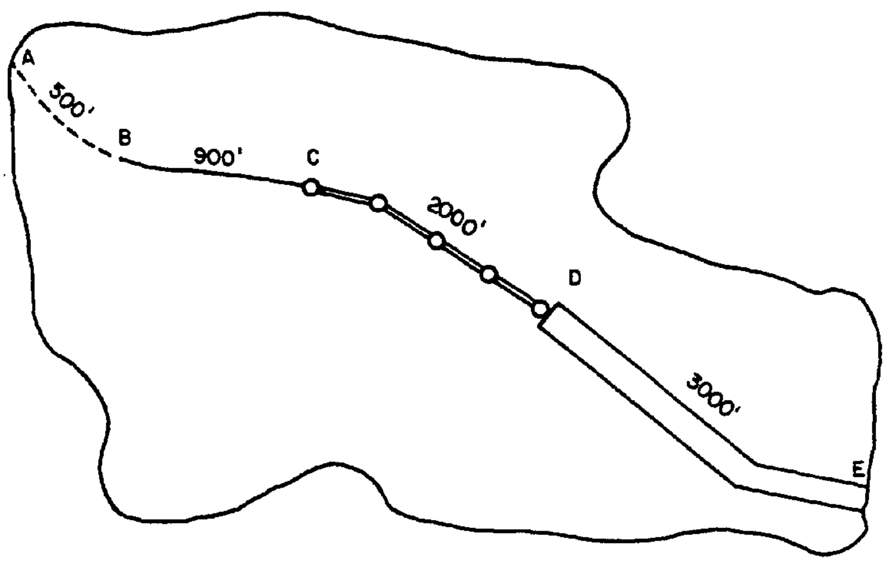
$$T_t = \frac{3500 \text{ft}}{4.4 \text{ ft /sec}} = 795 \text{ sec}$$

4. Total Time of Concentration

$$T_c = 714 + 854 + 795 = 2363 \text{ sec or } .66 \text{ HR}$$

The future conditions for this watershed are illustrated as follows.

| Reach | Description of Flow | Slope Percent | Length Feet |
|---|---|---|---|
| A to B | Overland (forest) | 7 | 500 |
| B to C | Overland (shallow gutter) | 2 | 900 |
| C to D | Storm drain with manhole covers, inlets, etc (n = 0.015; diameter 3 ft) | 1.5 | 2000 |
| D to E | Open channel, gunite, trapezoidal (b = 5; d = 3; z = 1:1; n = 0.019) | 0.5 | 3000 |



For the Future Condition

1. Compute overland flow travel time for reach A to B (this remains unchanged)

$$T_t = 714 \text{ sec}$$

2. Compute the overland flow for the reach B to C (street gutter). Again using Figure 52 for a slope of 2 percent, V = 2.8 ft/sec.

$$T_t = \frac{900 \text{ ft}}{2.8 \text{ ft/sec}} = 321 \text{ sec}$$

3. Compute the storm drain travel time, Reach C to D. Using Manning's Equation for pipe full velocity

$$V = \frac{1.49}{n} R_n^{2/3} S^{1/2} \quad ; \quad R_n = \frac{Dia}{4} = 0.75' \quad ; \quad S = 0.015$$

$$V = \frac{1.49}{0.015} (0.75)^{2/3} (0.015)^{1/2} = 10 \text{ ft/sec}$$

$$T_t = \frac{2000 \text{ ft}}{10 \text{ ft/sec}} = 200 \text{ sec}$$

4. Compute the open channel flow time for Reach D to E

$$V = \frac{1.49}{n} R_n^{2/3} S^{1/2} \qquad R_n = 1.78' \qquad S = 0.005$$

$$V = \frac{1.49}{.019} (1.78)^{2/3} (.005)^{1/2} = 8.2 \text{ ft/sec}$$

$$T_t = \frac{3000 \text{ ft}}{8.2 \text{ ft/sec}} = 366 \text{ sec}$$

5. Total Time of Concentration = 714 + 321 + 200 + 366 = 1601 secs

$$T_c = 0.44 \text{ hr}$$

The future condition has a time of concentration which is about 61 percent of the present condition.

Using the procedures presented above and the material about to be presented, the designer is able to quantify the effect of urbanization on both peak discharge and the design hydrograph. Two methods are presented in TR-55 for quantifying the effects of urbanization upon peak discharge. These are the Graphical Method and the Tabular Method.

8.3.4 Graphical Methods for Urban Peak Flow

This method discussed briefly in Section 6.3 is based on a Type II rainfall and is applicable when the runoff curve numbers can be assumed to be relatively uniform throughout the watershed and only a peak discharge is needed. The peak discharge for the watershed is determined for the present and future conditions from Figure 79, using the $T_c$ in hours, a 24-hour rainfall depth and the drainage area in square miles. The percentage change is then computed and applied to the present peak discharge.

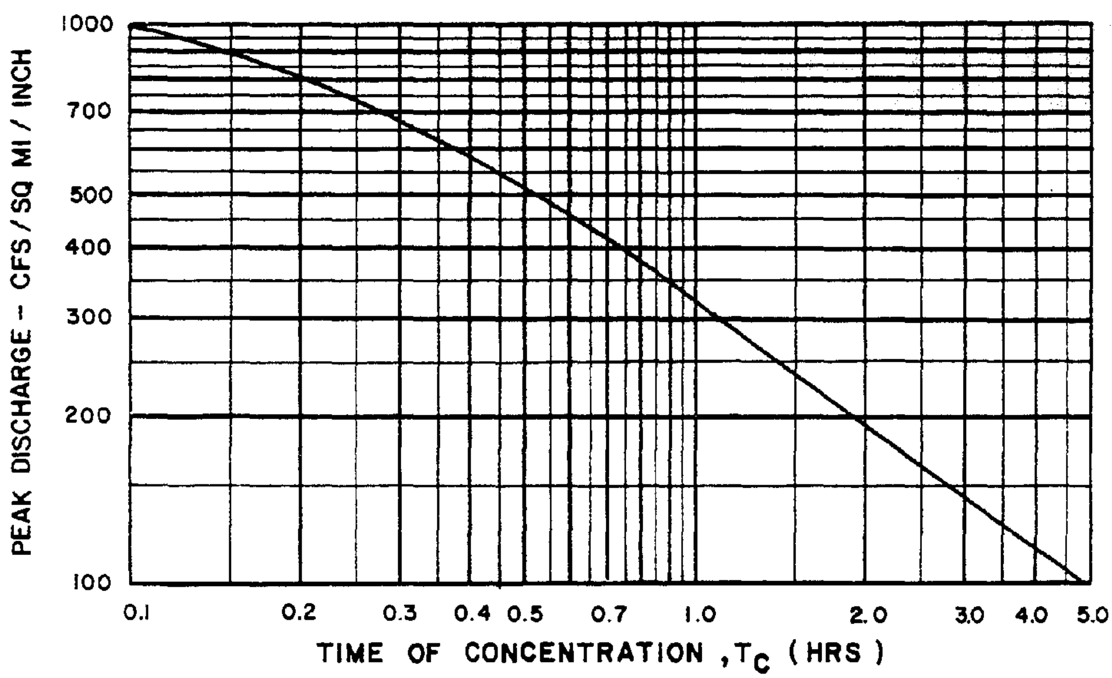



Figure 79. Peak Discharge as a Function of Time of Concentration for 24-Hour, Type II Storm Distribution

Note: The present peak discharge could have been determined using a different methodology and consequently could well differ from that given by the figure above. Since the interest is primarily in the relative effect of future urbanization on peak discharge, the methods of TR-55 are used to determine a percent change in peak discharge which can then be applied to the original estimate.

Example: An original estimate for the 100-year peak discharge for a 15 square mile watershed is 3250 CFS (92 CMS). What percentage increase can be expected due to urbanization?

The designer must determine the present conditions of the watershed and then assume what the future conditions will be. Sources of information which will be helpful in this regard are local zoning and planning agencies. The character of nearby watersheds which have undergone urbanization can also be evaluated to determine characteristic values within the region. For the present case the following data is assumed:

Drainage area = 15 square miles

CN (present) = 80

CN (future) = 85

$T_c$ (present) = 2.7 hours

$T_c$ (future) = 2.0 hours

$P_{24}$ (24-hour, 100-year rainfall depth) = 6.0 in

1. Determine present peak discharge using SCS methods for CN = 80 and P = 6.0 inches.

   First determine the direct runoff from Equation (8-10)

$$Q_{DR} = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{AND} \qquad S = \frac{1000}{CN} - 10$$

$$S = \frac{1000}{80} - 10 = 2.5$$

$$Q_{DR} = \frac{[6 - 0.2(2.5)]^2}{[6 + 0.8(2.5)]} = 3.78 \text{ inches}$$

Utilizing Figure 79 with $T_c$ = 2.7 hours.

Peak Discharge/sq mi/in = 153 CFS/mi$^2$/inch

$$Q = 153 \frac{\text{CSM}}{\text{inches}} \times 3.78 \text{ inches} \times 15 \text{ sq mi} = 8675 \text{ CFS (246 CMS)}$$

2. Determine future peak discharge using SCS Method for CN=85 and P=6

   Again, from Equation (8-10); $Q_{DR}$ = 4.30

   From Figure 79 for $T_c$ = 2.0 hours, Q = 190 CFS/sq mi/in

$$Q_{future} = Q_{peak\ discharge\ (future)} \text{ X Volume of Runoff}$$

$$Q = 190 \frac{\text{CSM}}{\text{inches}} \times 4.30 \text{ inches} \times 15 \text{ sq mi} = 12{,}255 \text{ CFS (345 CMS)}$$

3. Determine percent change

$$Q \times 100 = \frac{(12{,}255 - 8675)}{8675} = 41\ \%$$

4. Apply this percent change to original peak discharge estimate

$$\text{FUTURE } Q = 3250 \text{ CFS} \times 1.41 = 4591 \text{ CFS (130 CMS)}$$

   The effects of the estimated urbanization will be to increase the peak discharge from 3250 CFS (92 CMS) to 4591 CFS (131 CMS).

An alternate graphical method for computing modifications to peak discharge due to urbanization is presented in TR-55, SCS, 1975. The method is similar in concept to that described in Section 8.3.2 except that the adjustments for impervious area and channel improvements are applied to the peak discharge for future CN values.

The method is applicable to small drainage areas 1-2000 acres in size, and utilizes Figures 58, 59, and 60 which give a basic peak discharge rate for a 24 hour Type II storm for watersheds in natural conditions. The curves are applicable nationwide except for some portions of Washington, Oregon and California, SCS, 1975.

The modified discharge for urbanization is given by the relation

$$Q_{MOD} = Q \left[\text{FACTOR}_{IMP}\right]\left[\text{FACTOR}_{HLM}\right] \quad (8\text{-}12)$$

where $Q_{MOD}$ is the modified discharge due to urbanization in CFS/inch, Q is the discharge for future CN values in CFS/inch from Figures 58, 59, and 60. $\text{FACTOR}_{IMP}$ is an adjustment factor for percent impervious area given in Figure 80, and $\text{FACTOR}_{HLM}$ is an adjustment factor for percent of hydraulic length modified given in Figure 81.

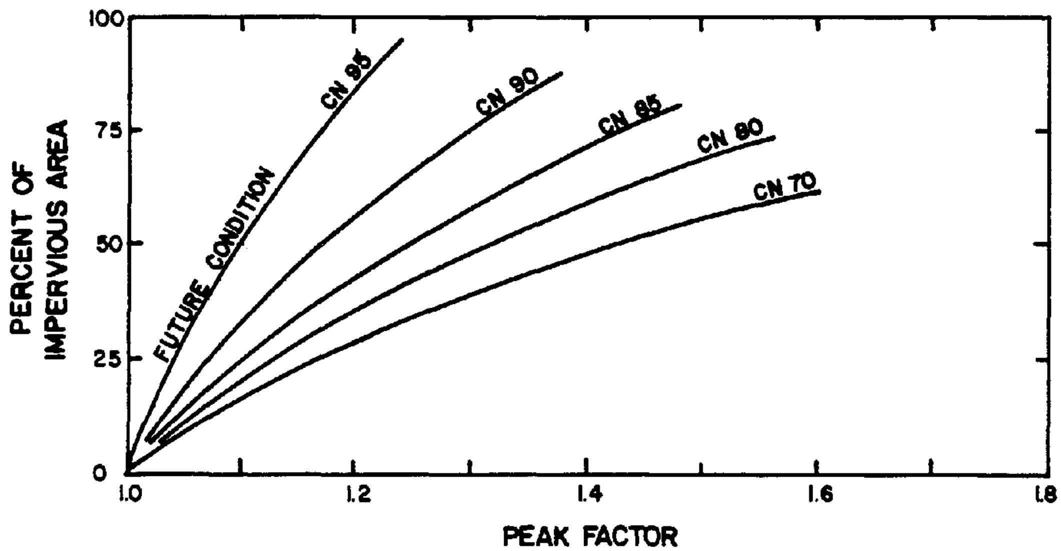



Figure 80. SCS Adjustment Factor for Percent Impervious Area

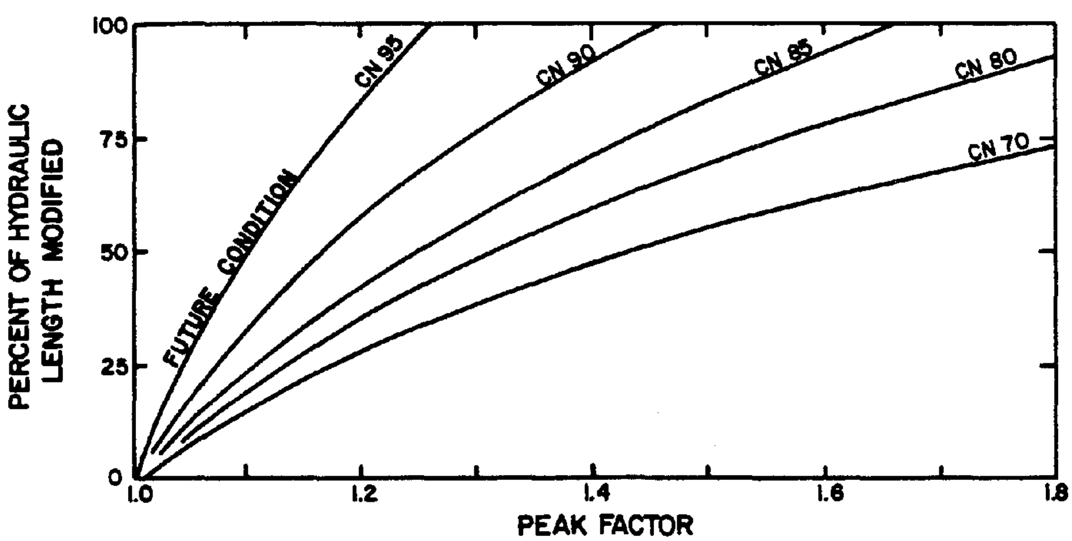



Figure 81. SCS Adjustment Factor for Percent of Modified Hydraulic Length

To illustrate the application of this procedure, consider the following example taken from TR-55.

<u>Example</u>

> A 300-acre watershed is to be developed. The runoff curve number for the proposed development is computed to be 80. Approximately 60 percent of the hydraulic length will be modified by the installation of street gutters and storm drains to the watershed outlet. Approximately 30 percent of the watershed will be impervious. The average watershed slope is estimated to be 4 percent. Compute the present-condition and anticipated future-condition peak discharge for a 50-year 24-hour storm event with 5 inches of rainfall. The present-condition runoff curve number is 75.

1. From Equation (8-10), the runoff for present and future conditions is computed.

$$Q_P = \frac{[5-0.2(3.3)]^2}{5+0.8(3.3)} = 2.45 \text{ inches}$$

$$Q_F = \frac{[5-0.2(2.5)]^2}{5+0.8(2.5)} = 2.89 \text{ inches}$$

2. From Figure 59 for moderate slope with CN = 75.

$$Q = 120 \text{ CFS /inch}$$

and

$$Q_P = (120)(2.45) = 294 \text{ CFS } (8.3 \text{ CMS})$$

3. From Figure 59 with CN = 80.

$$Q = 133 \text{ CFS /inches}$$

and

$$Q_P = (133)(2.89) = 384 \text{ CFS } (10.9 \text{ CMS})$$

4. For CN = 80, from Figure 80 with 30 percent impervious cover and from Figure 81 with 60 percent hydraulic length modifications,

$FACTOR_{IMP} = 1.16$

and

$FACTOR_{HLM} = 1.42$

5. The future peak flow from Equation (8-12) is

$$Q_{MOD} = 384(1.16)(1.42) = 633 \text{ CFS } (17.9 \text{ CMS})$$

6. The effect of the proposed development is to increase the peak flow from 294 CFS (8.3 CMS) to 633 CFS (17.9 CMS), an increase of 215 percent.

### 8.3.5 SCS Tabular Method

The tabular method is more applicable to larger watersheds than graphical methods, and can be used where watersheds are nonhomogeneous. Basically, the watershed in question is divided into homogeneous subareas. The runoff curve number, the time of concentration and the runoff for each subarea are determined for present and future conditions. With this information and Table 50, the peak discharge and runoff hydrograph for present and future conditions can be determined. Table 50 is a tabular representation of hydrographs from one square mile drainage areas routed through typical channels for a range of times of concentration and travel times. The computed values of time of concentration ($T_c$) and travel time ($T_t$) can be rounded to the nearest value used in Table 50 or, if more refinement is warranted, the discharges can be computed using the calculated $T_c$ and $T_t$ and interpolated between the $T_c$ and $T_t$ values shown in the table.

A more precise method would be to accurately model the present and future conditions of the watershed, determine a design hydrograph for each subarea and then route these design hydrographs to the watershed outlet. A complete model would be needed to provide definitive answers. Since highway designers usually assume future conditions, these models are rarely warranted in highway drainage design. The tabular method presented here is approximate and is used only to evaluate relative changes in stream discharge and hydrograph shape rather than provide detailed design hydrographs.

The tabular method is limited to conditions wherein changes in values of CN for the various subareas are not large and where the runoff volumes exceed 1.5 inches for CN's less than 60. For most conditions, however, the tabular method is sufficient to determine the effects of urbanization on peak flows for subareas up to about 20 square miles. To apply the SCS tabular method, the following information is needed to calculate the peak discharge.

1. Drainage area of each subarea

2. Time of concentration for each subarea

3. Time of travel for each routing reach

4. CN for each subarea

5. 24-hour rainfall for selected frequency

6. Runoff (in inches) for each subarea

Table 50. Tabular Discharges in CFS/sq mi/in for Type II Storm Distributions

TIME OF CONCENTRATION = 0.1 hours

| $T_t$ | Hydrograph Time in Hours 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 24 | 51 | 299 | 991 | 746 | 477 | 233 | 152 | 132 | 121 | 111 | 85 | 74 | 70 | 68 | 65 | 52 | 48 | 39 | 33 | 29 | 24 | 18 | 14 |
| 0.25 | 20 | 38 | 66 | 140 | 327 | 626 | 686 | 546 | 364 | 236 | 169 | 137 | 117 | 97 | 83 | 75 | 66 | 52 | 41 | 35 | 30 | 24 | 18 | 14 |
| 0.50 | 15 | 27 | 36 | 43 | 67 | 133 | 288 | 482 | 580 | 543 | 429 | 310 | 222 | 168 | 134 | 110 | 81 | 63 | 47 | 38 | 32 | 26 | 19 | 15 |
| 0.75 | 12 | 20 | 25 | 29 | 34 | 42 | 65 | 125 | 245 | 392 | 496 | 515 | 452 | 360 | 273 | 206 | 127 | 80 | 53 | 42 | 35 | 27 | 19 | 15 |
| 1.00 | 9 | 15 | 19 | 21 | 24 | 28 | 32 | 41 | 63 | 115 | 209 | 328 | 427 | 470 | 451 | 389 | 245 | 121 | 64 | 47 | 38 | 29 | 20 | 16 |
| 1.50 | 6 | 10 | 12 | 13 | 14 | 16 | 17 | 19 | 22 | 25 | 29 | 38 | 56 | 92 | 154 | 236 | 410 | 360 | 133 | 66 | 47 | 33 | 21 | 16 |
| 2.00 | 3 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 16 | 18 | 20 | 23 | 27 | 34 | 74 | 244 | 371 | 142 | 68 | 38 | 23 | 17 |
| 2.50 | 2 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 15 | 16 | 21 | 41 | 243 | 343 | 150 | 48 | 26 | 19 |
| 3.00 | 1 | 2 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 12 | 17 | 50 | 239 | 321 | 74 | 29 | 20 |
| 3.50 | 0 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 6 | 6 | 7 | 10 | 17 | 59 | 304 | 159 | 33 | 21 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 4 | 5 | 6 | 10 | 18 | 67 | 290 | 39 | 23 |



TIME OF CONCENTRATION = 0.2 hours

| $T_t$ | Hydrograph Time in Hours 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 23 | 47 | 208 | 509 | 796 | 641 | 424 | 245 | 170 | 138 | 121 | 104 | 85 | 75 | 71 | 68 | 56 | 49 | 40 | 34 | 29 | 24 | 18 | 14 |
| 0.25 | 18 | 34 | 49 | 91 | 196 | 419 | 603 | 627 | 486 | 341 | 235 | 173 | 138 | 114 | 96 | 83 | 70 | 55 | 43 | 36 | 31 | 25 | 18 | 15 |
| 0.50 | 14 | 24 | 32 | 37 | 50 | 87 | 181 | 341 | 490 | 545 | 497 | 397 | 296 | 219 | 167 | 133 | 92 | 67 | 49 | 39 | 33 | 26 | 19 | 15 |
| 0.75 | 11 | 18 | 23 | 26 | 30 | 36 | 49 | 84 | 161 | 284 | 409 | 491 | 481 | 422 | 340 | 263 | 157 | 89 | 56 | 43 | 36 | 27 | 19 | 15 |
| 1.00 | 9 | 14 | 18 | 20 | 22 | 25 | 29 | 35 | 48 | 79 | 143 | 240 | 347 | 426 | 452 | 427 | 299 | 147 | 69 | 49 | 39 | 29 | 20 | 16 |
| 1.50 | 5 | 9 | 11 | 12 | 13 | 14 | 16 | 18 | 20 | 23 | 26 | 32 | 43 | 67 | 110 | 176 | 330 | 399 | 159 | 72 | 50 | 33 | 22 | 17 |
| 2.00 | 3 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 15 | 16 | 18 | 21 | 24 | 29 | 56 | 192 | 363 | 168 | 75 | 40 | 24 | 18 |
| 2.50 | 1 | 3 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 15 | 19 | 33 | 200 | 337 | 174 | 51 | 26 | 19 |
| 3.00 | 0 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 11 | 15 | 40 | 203 | 316 | 82 | 29 | 20 |
| 3.50 | 0 | 0 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 9 | 16 | 46 | 300 | 180 | 34 | 22 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 6 | 9 | 16 | 53 | 286 | 41 | 24 |

Table 50. Tabular Discharges in CFS/sq mi/in for Type II Storm Distributions (Continued)

TIME OF CONCENTRATION = 0.3 hours

| $T_t$ | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
| 0 | 21 | 43 | 141 | 324 | 586 | 658 | 535 | 372 | 251 | 184 | 148 | 124 | 102 | 86 | 77 | 71 | 61 | 51 | 41 | 34 | 30 | 24 | 18 | 14 |
| 0.25 | 17 | 31 | 43 | 67 | 134 | 279 | 461 | 559 | 530 | 428 | 318 | 234 | 179 | 143 | 116 | 97 | 76 | 59 | 45 | 37 | 32 | 25 | 18 | 15 |
| 0.50 | 13 | 22 | 29 | 34 | 42 | 65 | 124 | 238 | 378 | 479 | 499 | 447 | 363 | 281 | 216 | 168 | 110 | 74 | 51 | 41 | 34 | 26 | 19 | 15 |
| 0.75 | 10 | 17 | 21 | 24 | 27 | 32 | 41 | 63 | 114 | 203 | 316 | 413 | 457 | 443 | 389 | 319 | 198 | 105 | 60 | 45 | 37 | 28 | 20 | 15 |
| 1.00 | 8 | 13 | 16 | 18 | 20 | 23 | 26 | 31 | 40 | 60 | 103 | 176 | 269 | 358 | 415 | 426 | 344 | 182 | 77 | 51 | 41 | 30 | 20 | 16 |
| 1.50 | 5 | 8 | 10 | 11 | 12 | 13 | 15 | 16 | 18 | 21 | 24 | 28 | 36 | 52 | 82 | 132 | 272 | 382 | 192 | 81 | 52 | 34 | 22 | 17 |
| 2.00 | 3 | 5 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 14 | 15 | 17 | 19 | 21 | 25 | 44 | 151 | 351 | 198 | 85 | 41 | 24 | 18 |
| 2.50 | 1 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 14 | 17 | 28 | 162 | 328 | 200 | 54 | 27 | 19 |
| 3.00 | 0 | 1 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 14 | 33 | 169 | 309 | 94 | 30 | 20 |
| 3.50 | 0 | 0 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 9 | 14 | 38 | 172 | 294 | 35 | 22 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 4 | 5 | 9 | 15 | 43 | 281 | 42 | 24 |

TIME OF CONCENTRATION = 0.4 hours

| $T_t$ | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
| 0 | 20 | 39 | 103 | 224 | 419 | 558 | 575 | 451 | 331 | 247 | 190 | 155 | 127 | 105 | 90 | 80 | 66 | 53 | 42 | 35 | 30 | 24 | 18 | 14 |
| 0.25 | 15 | 28 | 38 | 54 | 98 | 196 | 343 | 467 | 508 | 464 | 380 | 295 | 228 | 180 | 145 | 119 | 87 | 64 | 47 | 38 | 32 | 26 | 19 | 15 |
| 0.50 | 12 | 20 | 26 | 30 | 37 | 53 | 92 | 172 | 286 | 395 | 462 | 453 | 402 | 332 | 266 | 211 | 137 | 84 | 54 | 42 | 35 | 27 | 19 | 15 |
| 0.75 | 10 | 16 | 19 | 22 | 25 | 29 | 36 | 51 | 85 | 150 | 242 | 338 | 407 | 429 | 406 | 356 | 241 | 128 | 65 | 47 | 38 | 29 | 20 | 16 |
| 1.00 | 8 | 12 | 15 | 17 | 19 | 21 | 24 | 28 | 34 | 49 | 78 | 132 | 208 | 292 | 362 | 403 | 368 | 220 | 88 | 55 | 42 | 30 | 21 | 16 |
| 1.50 | 5 | 8 | 9 | 10 | 11 | 12 | 14 | 15 | 17 | 19 | 22 | 25 | 31 | 43 | 65 | 102 | 220 | 365 | 224 | 93 | 56 | 35 | 22 | 17 |
| 2.00 | 3 | 5 | 6 | 6 | 7 | 8 | 9 | 9 | 10 | 11 | 13 | 14 | 16 | 17 | 20 | 23 | 37 | 119 | 338 | 225 | 99 | 43 | 24 | 18 |
| 2.50 | 1 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 16 | 25 | 132 | 317 | 225 | 58 | 27 | 19 |
| 3.00 | 0 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 10 | 13 | 28 | 140 | 300 | 107 | 31 | 21 |
| 3.50 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 6 | 8 | 13 | 32 | 146 | 286 | 36 | 22 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 5 | 8 | 14 | 36 | 275 | 44 | 24 |

Table 50. Tabular Discharges in CFS/sq mi/in for Type II Storm Distributions (Continued)

TIME OF CONCENTRATION = 0.5 hours

| $T_t$ | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | | |
| 0 | 18 | 36 | 80 | 166 | 301 | 433 | 496 | 474 | 395 | 309 | 242 | 194 | 158 | 130 | 109 | 94 | 75 | 57 | 43 | 36 | 31 | 25 | 18 | 15 |
| 0.25 | 15 | 26 | 37 | 52 | 94 | 172 | 277 | 372 | 425 | 424 | 383 | 326 | 270 | 221 | 182 | 150 | 107 | 73 | 49 | 39 | 33 | 26 | 19 | 15 |
| 0.50 | 12 | 20 | 25 | 30 | 38 | 58 | 101 | 169 | 252 | 327 | 374 | 385 | 366 | 329 | 285 | 241 | 169 | 103 | 59 | 44 | 36 | 27 | 19 | 15 |
| 0.75 | 9 | 15 | 19 | 22 | 25 | 30 | 41 | 63 | 103 | 162 | 229 | 292 | 335 | 354 | 348 | 325 | 255 | 157 | 77 | 50 | 39 | 29 | 20 | 16 |
| 1.00 | 7 | 12 | 15 | 17 | 19 | 21 | 25 | 31 | 43 | 66 | 103 | 153 | 210 | 264 | 304 | 327 | 317 | 231 | 109 | 61 | 44 | 31 | 21 | 16 |
| 1.50 | 5 | 8 | 9 | 10 | 11 | 12 | 14 | 15 | 17 | 20 | 24 | 31 | 43 | 63 | 92 | 129 | 214 | 295 | 224 | 115 | 65 | 36 | 23 | 17 |
| 2.00 | 3 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 16 | 19 | 23 | 30 | 58 | 143 | 271 | 216 | 120 | 46 | 25 | 18 |
| 2.50 | 1 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 14 | 18 | 39 | 150 | 253 | 209 | 71 | 28 | 19 |
| 3.00 | 0 | 1 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 10 | 15 | 48 | 154 | 239 | 126 | 32 | 21 |
| 3.50 | 0 | 0 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 8 | 16 | 56 | 155 | 227 | 38 | 23 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 4 | 5 | 9 | 19 | 63 | 217 | 52 | 25 |

TIME OF CONCENTRATION = 0.75 hours

| $T_t$ | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | | |
| 0 | 15 | 29 | 57 | 98 | 163 | 248 | 329 | 375 | 388 | 369 | 325 | 276 | 232 | 195 | 165 | 142 | 107 | 76 | 51 | 39 | 33 | 26 | 19 | 15 |
| 0.25 | 12 | 21 | 29 | 39 | 61 | 100 | 158 | 227 | 291 | 336 | 355 | 348 | 321 | 285 | 247 | 212 | 156 | 103 | 62 | 44 | 36 | 27 | 19 | 15 |
| 0.50 | 10 | 16 | 21 | 24 | 29 | 41 | 63 | 100 | 150 | 208 | 263 | 305 | 327 | 329 | 314 | 288 | 226 | 147 | 79 | 52 | 40 | 29 | 20 | 16 |
| 0.75 | 8 | 13 | 16 | 18 | 20 | 24 | 30 | 43 | 65 | 98 | 142 | 192 | 239 | 278 | 303 | 311 | 286 | 208 | 107 | 63 | 45 | 31 | 21 | 16 |
| 1.00 | 6 | 10 | 13 | 14 | 15 | 17 | 20 | 24 | 31 | 44 | 65 | 95 | 134 | 177 | 220 | 256 | 294 | 264 | 149 | 81 | 53 | 33 | 21 | 16 |
| 1.50 | 4 | 6 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 16 | 19 | 23 | 31 | 42 | 60 | 83 | 147 | 269 | 248 | 152 | 85 | 40 | 23 | 17 |
| 2.00 | 2 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 14 | 16 | 18 | 23 | 39 | 97 | 251 | 235 | 153 | 56 | 26 | 19 |
| 2.50 | 1 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 15 | 28 | 107 | 218 | 236 | 91 | 29 | 20 |
| 3.00 | 0 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 12 | 33 | 113 | 225 | 153 | 34 | 22 |
| 3.50 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 7 | 13 | 39 | 117 | 215 | 44 | 24 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 4 | 7 | 15 | 45 | 207 | 63 | 26 |

Table 50. Tabular Discharges in CFS/sq mi/in for Type II Storm Distributions (Continued)

TIME OF CONCENTRATION = 1.0 hours

| $T_t$ | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
| 0 | 13 | 24 | 45 | 66 | 107 | 155 | 211 | 258 | 301 | 313 | 316 | 301 | 277 | 247 | 217 | 188 | 146 | 102 | 64 | 46 | 36 | 27 | 19 | 15 |
| 0.25 | 10 | 18 | 24 | 32 | 45 | 68 | 102 | 146 | 193 | 238 | 272 | 293 | 299 | 293 | 275 | 252 | 200 | 139 | 81 | 54 | 41 | 29 | 20 | 16 |
| 0.50 | 8 | 14 | 17 | 20 | 24 | 32 | 46 | 68 | 99 | 136 | 178 | 219 | 251 | 274 | 284 | 283 | 254 | 187 | 105 | 65 | 47 | 31 | 21 | 16 |
| 0.75 | 7 | 11 | 13 | 15 | 17 | 20 | 25 | 33 | 46 | 67 | 94 | 128 | 165 | 202 | 233 | 256 | 273 | 236 | 140 | 82 | 55 | 33 | 21 | 16 |
| 1.00 | 5 | 9 | 11 | 12 | 13 | 15 | 17 | 20 | 25 | 33 | 46 | 65 | 90 | 121 | 154 | 187 | 240 | 262 | 183 | 107 | 66 | 37 | 22 | 17 |
| 1.50 | 3 | 5 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 14 | 16 | 19 | 24 | 31 | 43 | 58 | 103 | 185 | 244 | 181 | 110 | 48 | 24 | 18 |
| 2.00 | 2 | 3 | 4 | 4 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 13 | 15 | 18 | 29 | 69 | 182 | 230 | 178 | 70 | 27 | 19 |
| 2.50 | 1 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 12 | 21 | 77 | 178 | 219 | 114 | 31 | 21 |
| 3.00 | 0 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 10 | 25 | 83 | 210 | 172 | 39 | 22 |
| 3.50 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 6 | 11 | 29 | 88 | 202 | 52 | 25 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 4 | 6 | 12 | 33 | 195 | 77 | 28 |

TIME OF CONCENTRATION = 1.25 hours

| $T_t$ | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
| 0 | 11 | 21 | 37 | 51 | 79 | 107 | 147 | 187 | 219 | 249 | 264 | 271 | 267 | 256 | 241 | 219 | 177 | 128 | 81 | 56 | 42 | 29 | 20 | 16 |
| 0.25 | 9 | 15 | 21 | 27 | 36 | 53 | 74 | 103 | 137 | 172 | 205 | 231 | 249 | 259 | 259 | 253 | 223 | 167 | 102 | 67 | 48 | 31 | 21 | 16 |
| 0.50 | 7 | 12 | 15 | 17 | 21 | 27 | 37 | 51 | 72 | 98 | 128 | 160 | 190 | 216 | 235 | 247 | 251 | 209 | 130 | 82 | 56 | 34 | 21 | 16 |
| 0.75 | 6 | 9 | 12 | 13 | 15 | 17 | 21 | 27 | 36 | 50 | 69 | 93 | 120 | 149 | 177 | 202 | 235 | 242 | 165 | 103 | 67 | 38 | 22 | 17 |
| 1.00 | 4 | 7 | 9 | 10 | 11 | 13 | 14 | 17 | 21 | 27 | 36 | 49 | 66 | 88 | 113 | 139 | 190 | 236 | 200 | 130 | 83 | 43 | 23 | 17 |
| 1.50 | 3 | 5 | 6 | 6 | 7 | 8 | 8 | 9 | 10 | 12 | 14 | 16 | 20 | 25 | 33 | 44 | 76 | 142 | 223 | 195 | 131 | 58 | 26 | 18 |
| 2.00 | 1 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 10 | 11 | 13 | 15 | 24 | 52 | 143 | 212 | 189 | 86 | 29 | 20 |
| 2.50 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 10 | 17 | 58 | 143 | 201 | 132 | 35 | 21 |
| 3.00 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 6 | 9 | 20 | 64 | 143 | 196 | 45 | 23 |
| 3.50 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 4 | 5 | 9 | 23 | 68 | 190 | 62 | 26 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 3 | 5 | 10 | 26 | 184 | 91 | 30 |

Table 50. Tabular Discharges in CFS/sq mi/in for Type II Storm Distributions (Continued)

TIME OF CONCENTRATION = 1.5 hours

| $T_t$ | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
| 0 | 10 | 18 | 31 | 42 | 57 | 81 | 105 | 133 | 164 | 192 | 209 | 227 | 235 | 236 | 236 | 225 | 201 | 153 | 99 | 68 | 50 | 32 | 20 | 16 |
| 0.25 | 8 | 13 | 17 | 22 | 30 | 41 | 57 | 76 | 99 | 125 | 153 | 178 | 199 | 215 | 225 | 230 | 224 | 188 | 122 | 82 | 58 | 36 | 21 | 16 |
| 0.50 | 6 | 10 | 13 | 15 | 18 | 22 | 30 | 40 | 54 | 72 | 94 | 118 | 143 | 167 | 188 | 204 | 224 | 214 | 152 | 99 | 68 | 39 | 22 | 17 |
| 0.75 | 5 | 8 | 10 | 11 | 13 | 15 | 18 | 22 | 29 | 39 | 52 | 69 | 89 | 111 | 134 | 157 | 194 | 219 | 182 | 122 | 82 | 44 | 23 | 17 |
| 1.00 | 4 | 6 | 8 | 9 | 10 | 11 | 12 | 14 | 17 | 22 | 29 | 38 | 50 | 66 | 84 | 105 | 148 | 198 | 214 | 150 | 100 | 50 | 24 | 18 |
| 1.50 | 2 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 12 | 14 | 17 | 21 | 26 | 34 | 58 | 109 | 191 | 204 | 149 | 70 | 28 | 19 |
| 2.00 | 1 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 8 | 10 | 11 | 13 | 19 | 40 | 112 | 184 | 197 | 102 | 33 | 20 |
| 2.50 | 0 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 9 | 14 | 45 | 114 | 190 | 147 | 40 | 22 |
| 3.00 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 7 | 16 | 49 | 115 | 184 | 53 | 25 |
| 3.50 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 4 | 8 | 18 | 53 | 178 | 74 | 28 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 4 | 8 | 21 | 174 | 105 | 34 |

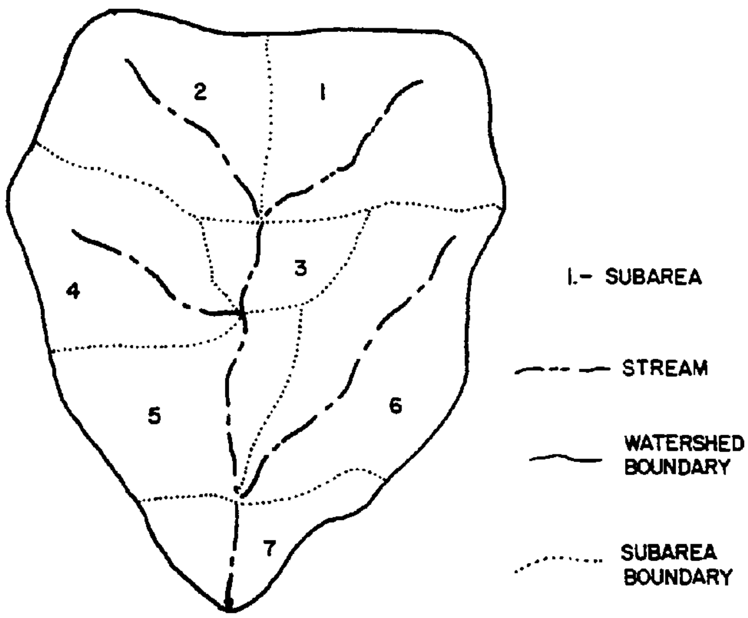
TIME OF CONCENTRATION = 2.0 hours

| $T_t$ | Hydrograph Time in Hours | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 11.0 | 11.5 | 11.7 | 11.8 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 12.9 | 13.0 | 13.2 | 13.5 | 14.0 | 14.5 | 15.0 | 16.0 | 18.0 | 20.0 |
| 0 | 7 | 14 | 22 | 30 | 38 | 49 | 64 | 80 | 95 | 114 | 133 | 152 | 165 | 175 | 184 | 192 | 190 | 176 | 129 | 93 | 68 | 41 | 23 | 17 |
| 0.25 | 6 | 10 | 13 | 17 | 22 | 28 | 37 | 47 | 61 | 75 | 91 | 108 | 126 | 143 | 157 | 168 | 185 | 189 | 153 | 109 | 79 | 46 | 24 | 17 |
| 0.50 | 5 | 8 | 10 | 11 | 13 | 17 | 21 | 27 | 35 | 45 | 57 | 71 | 86 | 103 | 119 | 135 | 162 | 186 | 172 | 129 | 92 | 52 | 26 | 18 |
| 0.75 | 4 | 6 | 8 | 8 | 10 | 11 | 13 | 16 | 21 | 26 | 34 | 43 | 55 | 67 | 82 | 97 | 129 | 166 | 183 | 149 | 109 | 59 | 27 | 18 |
| 1.00 | 3 | 5 | 6 | 7 | 7 | 8 | 9 | 11 | 13 | 16 | 20 | 26 | 33 | 42 | 52 | 64 | 92 | 136 | 180 | 167 | 127 | 68 | 29 | 19 |
| 1.50 | 1 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 12 | 15 | 18 | 23 | 37 | 68 | 135 | 175 | 163 | 93 | 34 | 21 |
| 2.00 | 1 | 1 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 10 | 14 | 26 | 71 | 133 | 170 | 127 | 42 | 23 |
| 2.50 | 0 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 7 | 11 | 29 | 74 | 132 | 166 | 53 | 26 |
| 3.00 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 3 | 3 | 4 | 5 | 12 | 32 | 76 | 162 | 71 | 30 |
| 3.50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 3 | 6 | 13 | 35 | 158 | 95 | 35 |
| 4.00 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 1 | 1 | 1 | 2 | 3 | 6 | 14 | 80 | 155 | 43 |

As an illustration of the tabular method of computation, the following example is taken from TR-55, SCS, 1975.

Example

A developer plans to develop subareas 5, 6, and 7 shown in the sketch below. The township planning board, before accepting his proposal, wants to know what effect the development would have on the 100-year discharge at the downstream end of subarea 7.




1. Develop a table similar to that shown below which provides a summary of all the basic data required in the tabular hydrograph method.

Basic data used in Example of Tabular Method

| Sub-area | Drainage Area (mi$^2$) | Time of Concentration (hrs) | | Runoff Curve Number | | Runoff[1] (in) | | Travel time[2] (hrs) | |
|---|---|---|---|---|---|---|---|---|---|
| | | Pres. | Fut. | Pres. | Fut. | Pres. | Fut. | Pres. | Fut. |
| 1 | 0.3 | 1.50 | 1.50 | 65 | 65 | 2.35 | 2.35 | - | - |
| 2 | 0.2 | 1.25 | 1.25 | 70 | 70 | 2.80 | 2.80 | - | - |
| 3 | 0.1 | 0.50 | 0.50 | 75 | 75 | 3.28 | 3.28 | 0.25 | 0.25 |
| 4 | 0.25 | 0.75 | 0.75 | 70 | 70 | 2.80 | 2.80 | - | - |
| 5 | 0.2 | 1.50 | 1.50 | 75 | 85 | 3.28 | 4.31 | 1.25 | 1.00 |
| 6 | 0.4 | 1.50 | 1.00 | 70 | 75 | 2.80 | 3.28 | - | - |
| 7 | 0.2 | 1.25 | 0.75 | 75 | 90 | 3.28 | 4.85 | 0.75 | 0.50 |

[1] From Equation (8-10) for P = 6 inches

[2] Travel time through the reach for the corresponding subarea.

2. Develop a flood routing summary table similar to that shown in Table 51 for present and future conditions. The $T_t$ for each subarea is the total travel time for that subarea through the watershed to the point of interest (end of subarea 7). The hydrograph coordinates under time-hours for each subarea are computed using the appropriate values from Table 50 and the equation $q = q_p(DA)(Q_{DR})$ where q is the hydrograph discharge coordinate in CFS, $q_p$ is in csm/in (cubic feet per second per square mile per inch of runoff), DA is the drainage area in sq mi, and $Q_{DR}$ is the runoff in inches.

   Using subarea 4 as an example, for $T_c$ = 0.75 hrs and $T_t$ = 2.00 hrs (the travel time through subareas 5 and 7) the routed peak of subarea 4 appears at the outlet of subarea 7 at 14.0 hours and is 251 CFS/mi$^2$/in Therefore, the peak discharge is: q = 251(.25)(2.80) = 176 CFS (5 CMS).

3. In order to develop a composite hydrograph at the end of subarea 7, the hydrographs from each subarea are summed. This method provides a means of adjusting the timing of each hydrograph to allow for the travel time ($T_t$) from the individual watershed to the point in question. The summary table shows how the present and future discharges are estimated. The effect of the urban development is to increase the 100-year peak discharge from 752 to 894 CFS (21.3 - 25.3 CMS) or approximately 20 percent.

4. Using the flows from the summary table, the composite hydrographs at the end of subarea 7 are plotted in Figure 82 for both present and future conditions.

## 8.4 Channelization

Channelization is the process of modifying the hydraulic conveyance of a natural watershed. This is usually done to improve the hydraulic efficiency of the main channel and tributaries and thereby alleviate localized flooding problems. On the other hand, the results of channelization are usually reflected in an increase in the peak discharge and a decrease in the time to peak of the runoff hydrograph.

The effects of channelization have been incorporated into several of the methods described above for inclusion of urban effects. The USGS Basin Development Factor is determined primarily from channel improvements and the methods of TR-55 provide peak flow and time of concentration adjustments based on the percent of channel improvements. The methods of channel routing presented in Section 7.1 can also be used to evaluate the effects of channelization as was illustrated by the example presented in that section.

Various urban studies such as that by Liscum and Massey, 1980, have shown that the impacts of channelization on flood characteristics may be as significant as the encroachment of impervious cover. Therefore, the designer

Table 51. Discharge Summary for SCS Tabular Method

| Present Conditions | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sub-area | $T_c$ | $T_t$ | Drainage Area | Rain-fall | CN | Run-off | 11.0 hr | 12.0 hr | 12.5 hr | 13.0 hr | 13.2 hr | 13.5 hr | 14.0 hr | 14.5 hr | 15.0 hr | 16.0 hr | 18.0 hr | 20.0 hr |
| | Hr | Hr | $Mi^2$ | In | | In | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS |
| 1[1] | 1.50 | 2.25 | 0.30 | 6 | 65 | 2.35 | 1 | 2 | 4 | 7 | 10 | 19 | 55 | 105 | 136 | 88 | 26 | 15 |
| 2[1] | 1.25 | 2.25 | 0.20 | 6 | 70 | 2.80 | 1 | 2 | 4 | 6 | 10 | 19 | 56 | 99 | 109 | 61 | 18 | 11 |
| 3 | 0.50 | 2.00 | 0.10 | 6 | 75 | 3.28 | 1 | 3 | 4 | 10 | 19 | 47 | 89 | 71 | 39 | 15 | 8 | 6 |
| 4 | 0.75 | 2.00 | 0.25 | 6 | 70 | 2.80 | 1 | 5 | 8 | 16 | 27 | 68 | 176 | 165 | 107 | 39 | 18 | 13 |
| 5 | 1.50 | 0.75 | 0.20 | 6 | 75 | 3.28 | 3 | 10 | 34 | 103 | 127 | 144 | 119 | 80 | 54 | 29 | 15 | 11 |
| 6 | 1.50 | 0.75 | 0.40 | 6 | 70 | 2.80 | 6 | 17 | 58 | 176 | 217 | 245 | 204 | 137 | 92 | 49 | 26 | 19 |
| 7 | 1.25 | 0.00 | 0.20 | 6 | 75 | 3.28 | 7 | 70 | 173 | 144 | 116 | 84 | 53 | 37 | 28 | 19 | 13 | 10 |
| Total (Composite hydrograph at end of subarea 7) | | | | | | | 20 | 109 | 285 | 462 | 526 | 626 | 752 | 694 | 565 | 300 | 124 | 85 |

| Future Conditions | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sub-area | $T_c$ | $T_t$ | Drainage Area | Rain-fall | CN | Run-off | 11.0 hr | 12.0 hr | 12.5 hr | 13.0 hr | 13.2 hr | 13.5 hr | 14.0 hr | 14.5 hr | 15.0 hr | 16.0 hr | 18.0 hr | 20.0 hr |
| | Hr | Hr | $Mi^2$ | In | | In | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS | CFS |
| 1[1] | 1.50 | 1.75 | 0.30 | 6 | 65 | 2.35 | 1 | 4 | 7 | 17 | 27 | 53 | 107 | 137 | 122 | 60 | 21 | 13 |
| 2[1] | 1.25 | 1.75 | 0.20 | 6 | 70 | 2.80 | 1 | 3 | 6 | 17 | 28 | 54 | 102 | 114 | 90 | 40 | 15 | 11 |
| 3 | 0.50 | 1.50 | 0.10 | 6 | 75 | 3.28 | 2 | 4 | 8 | 42 | 70 | 97 | 73 | 38 | 21 | 12 | 8 | 6 |
| 4 | 0.75 | 1.50 | 0.25 | 6 | 70 | 2.80 | 3 | 8 | 13 | 58 | 103 | 188 | 174 | 106 | 60 | 28 | 16 | 12 |
| 5 | 1.50 | 0.50 | 0.20 | 6 | 85 | 4.31 | 5 | 19 | 81 | 176 | 193 | 184 | 131 | 85 | 59 | 34 | 19 | 15 |
| 6 | 1.00 | 0.50 | 0.40 | 6 | 75 | 3.28 | 10 | 42 | 234 | 371 | 333 | 245 | 138 | 85 | 62 | 41 | 28 | 21 |
| 7 | 0.75 | 0.00 | 0.20 | 6 | 90 | 4.85 | 15 | 241 | 315 | 138 | 104 | 73 | 49 | 38 | 32 | 25 | 18 | 15 |
| Total(Composite hydrograph at end of subarea 7) | | | | | | | 37 | 321 | 664 | 819 | 858 | 894 | 774 | 603 | 446 | 240 | 125 | 93 |

[1]Discharges for these areas are computed from interpolated csm/in (cubic feet per second per square mile per inch of runoff) values from Table 50

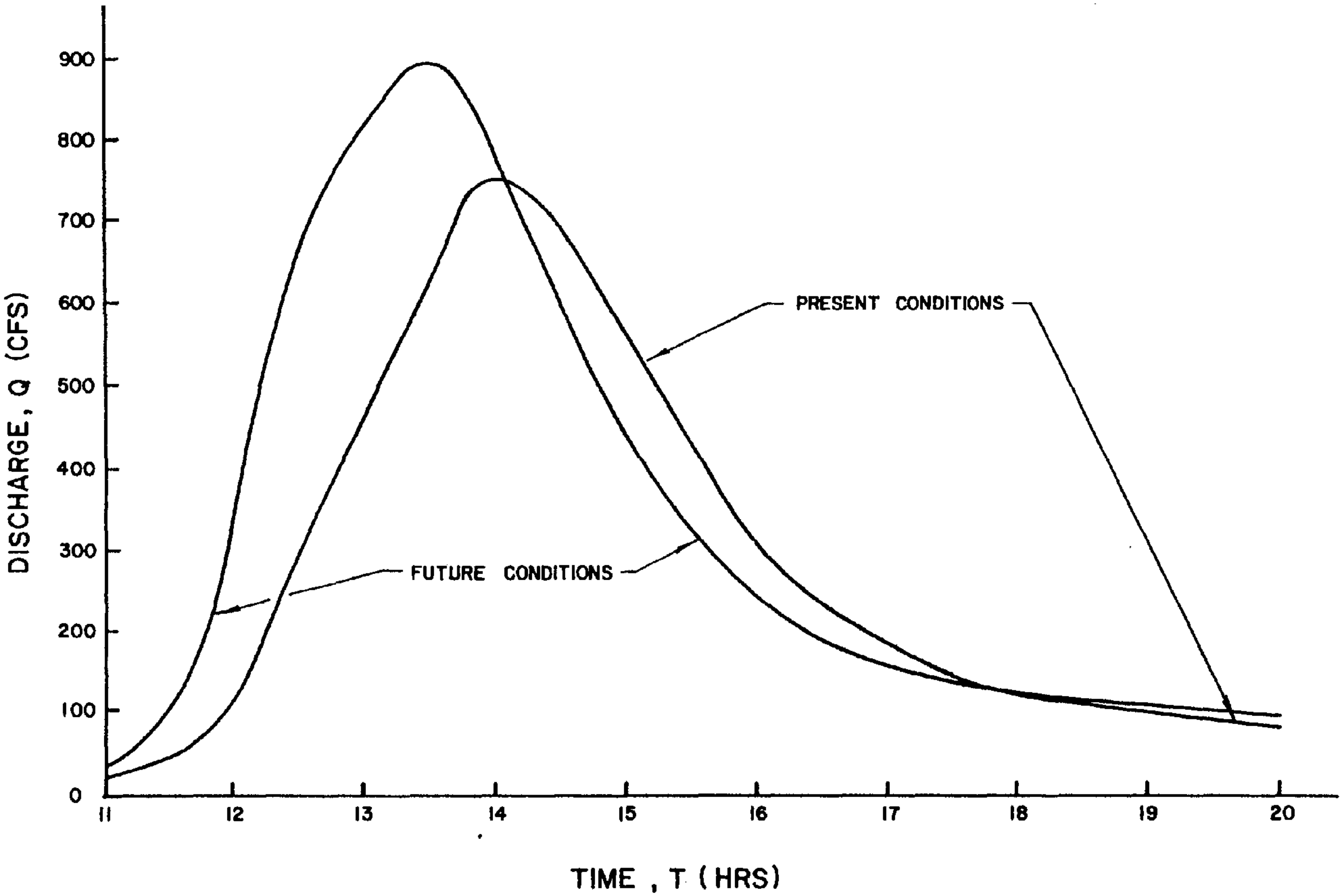



Figure 82. SCS Composite Hydrographs for Present and Future Conditions

must be able to evaluate the effects of channelization work done by others on highway design as well as any improvements made in conjunction with highway construction.

### 8.5 Detention Storage

Temporary in-channel or detention storage usually reduces peak discharges. Unfortunately, there is no simple way to determine the effect of detention storage at a specified urban site. The reservoir- and channel-routing techniques discussed in Section 7.0 must be used to make assessments of these quantities.

### 8.6 Diversions and Dam Construction

The highway designer needs to be aware of the construction or planned construction of diversions or dams on the watershed he is dealing with because these works will significantly affect the magnitude and character of the runoff reaching the highway crossing. The designer should make a point to keep informed of proposed projects being studied by the various water resources agencies active in their part of the country. A few of the most active agencies have been listed in Appendix C. Local agencies such as power utilities, irrigation boards and water supply companies should be canvassed whenever a major highway drainage structure is designed. The methods of channel and reservoir routing must be used to assess the effects such projects will have on highway drainage.

### 8.7 Natural Disasters

It was pointed out earlier that highways are permanent structures. Although it is rarely economically feasible to design a highway drainage structure to convey extremely rare discharges unimpeded, the occurrence of such events should not be ignored. Many highway departments have adopted policies which require drainage structures to be designed for a specified recurrence interval, but checked for a higher recurrence interval (often the 100-year discharge, the overtopping flood or the flood of record). It was shown in Section 4.0 that there is a 40 percent chance that during a 50-year period a drainage structure will be subjected to a discharge equal to or greater than the 100-year discharge. The longer a structure is in place the more likely it will be subjected to a discharge much greater than the design discharge. This risk can be quantified based upon the laws of probability and this is discussed in more detail in Section 9.0 on risk analysis. Checking for the effects of a rare event is one method of focusing the designers attention upon this aspect of design. However, factors other than discharge must be evaluated. These include the occurrence of earthquakes, forest fires, dam breaks and other unlikely but possible events. The designer needs to assess the vulnerability of the particular site with respect to the effects of these occurrences. It is very difficult to assign a recurrence interval to such natural disasters, but their impacts can sometimes be modeled.

The effects of forest fires upon the rainfall runoff response of a watershed can be estimated based upon previous experience. The U.S. Forest Service can be contacted to provide guidance in this area. The effects of dam breaks have been studied by the National Weather Service, and the NWS is available for consultation and guidance.

Often, after a natural disaster strikes, detailed studies of the effects are made and reports generated which can serve as guidance to the designer. The National Weather Service, the U.S. Geological Survey and the Corps of Engineers are the primary sources of such reports.

# 9.0 RISK ANALYSIS

Throughout this manual, techniques and procedures have been presented to determine the hydrologic parameters needed for design of highway stream crossings. Emphasis has been on determination of peak discharges and hydrographs because these are among the most important design parameters. In the previous section, it was pointed out that highway drainage structures are permanent and their design should take this into account. This section presents a technique for quantifying the risk that a given design discharge will be exceeded during the design life of the structure.

## 9.1 Evaluation of Risk

In Sections 4.0, 5.0 and 8.0, methods were presented for determining the peak discharge for a given recurrence interval. Recurrence interval (or return period) was defined in Equation (4-7) as the reciprocal of the probability that a particular peak discharge will be exceeded in any one year. If a drainage structure has a design life of 50 years, the question arises as to the risk that a particular design discharge will be exceeded at least once during that 50-year period. The lower the probability of the design discharge then the lower the risk of this happening during the design life. On the other hand, the longer the structure is not subjected to the design storm, the higher the risk over the remainder of its life. This can be quantified by the following equation: (previously given as Equation (4-10))

$$R = 1 - \left(1 - \frac{1}{T_r}\right)^m \qquad (9\text{-}1)$$

where R is the risk of the design discharge being exceeded at least once in the design life, Tr is the recurrence interval of the design discharge, and m is the design life in years.

This equation is tabulated in Table 52 as a function of recurrence interval and design life. An abbreviated form of this table, in slightly different form, was given earlier as Table 6.

Table 52. Tabulation of Risk of at Least One Exceedance During Design Life as a function of Recurrence Interval and Design Life

| Recurrence Interval | DESIGN LIFE-YEARS 2 | 5 | 10 | 25 | 50 | 100 |
|---|---|---|---|---|---|---|
| 2 | .75 | .97 | ≃ 1.00 | ≃ 1.00 | ≃ 1.00 | ≃ 1.00 |
| 5 | .36 | .67 | .89 | ≃ 1.00 | ≃ 1.00 | ≃ 1.00 |
| 10 | .19 | .41 | .65 | .93 | .99 | ≃ 1.00 |
| 25 | .08 | .18 | .34 | .64 | .87 | .98 |
| 50 | .04 | .10 | .18 | .40 | .64 | .87 |
| 100 | .02 | .05 | .10 | .22 | .39 | .63 |
| 500 | .004 | .01 | .02 | .05 | .10 | .18 |
| 1000 | .002 | .005 | .01 | .02 | .05 | .10 |

Example: What is the risk that for a design life of 50 years at least one discharge greater than the 100-year discharge will occur?

From Table 52, R = 0.39, or there is a 39 percent chance the 100-year discharge will be exceeded over the project's design life.

Another way to use Equation (9-1) is to determine what recurrence interval is associated with a selected value of risk.

Example: If the designer decides that he can only accept a 5 percent chance of a roadway being overtopped during its 50-year design life, what is the recurrence interval of the overtopping discharge?

Rearranging Equation (9-1) gives

$$T_r = \frac{1}{1-(1-R)^{1/m}} \quad (9\text{-}2)$$

or

$$T_r = \frac{1}{1-(1-.05)^{1/50}} = 975 \text{ YRS}$$

To reduce the risk of overtopping to 5 percent over the 50-year design life of the project, the drainage structure must be designed for a peak flow with a recurrence interval of 975 years.

Equation (9-2) puts the establishment of reasonable design parameters in a better perspective. Obviously, it is not possible to reliably estimate discharges with very large recurrence intervals such as above using the normal statistical methods presented earlier. The available records are not long enough to allow valid statistical analyses. Therefore, if the designer wishes to provide for very low levels of risk for certain events, such as overtopping, it is necessary to utilize more sophisticated methods of modeling the hydrology of the watershed in order to define the rare discharges involved. Such techniques are beyond the scope of this manual.

## 9.2 Uncertainty

Risk as defined above is associated with the probability of exceedance of a selected design value. Risk is inherent in nature and exists even if there were complete and correct definition of the probability distribution of the random variables (peak discharges). Uncertainty is a term sometimes used to account for the estimates of probabilities made from the limited samples of data used by the designer to determine flood peaks of given frequencies. Uncertainty can only be reduced by eliminating sources of error and using improved data collection and analysis. The combination of risk and uncertainty as defined above is the total risk, or simply risk, and is estimated from the probabilities of exceedance and non-exceedance using the available data sample.

The above section has raised question of the reliability of estimates of design parameters. How good are the estimates? How good do they have to be? The answers to these questions depend upon a number of factors.

The reliability of estimates of peak discharge depends upon the length of record available and also upon the assumed frequency distribution. Other sources of error in the statistical estimates of peak discharge include outliers, mixed populations, and inaccurate data. Methods were presented in Section 4.3.7 to evaluate these sources of error and to adjust for many of them. If it can be assumed that all errors have been eliminated and that the chosen frequency distribution exactly fits the frequency distribution of the population of peak discharges, then the reliability of the estimates will depend only upon the length of record available. The longer the record the better the estimate. The reliability of the estimate is then measured by the confidence limits presented in the discussion of hydrologic statistics, Sec. 4.3.6.2.

The equations necessary to compute confidence levels are somewhat tedious to apply. Table 53 gives approximate values for the reliability of estimates of peak discharge for various lengths of record and return periods.

Table 53. Approximate Values for the Reliability of Estimates of Peak Discharge for Various Lengths of Record and Return Periods

| FOR LENGTH OF RECORD = 10 years | | | |
|---|---|---|---|
| | Percent Error Allowed | | |
| Tr | 10% | 25% | 50% |
| 2 Yrs | 47 | 88 | 99 |
| 5 Yrs | 48 | 86 | 98 |
| 10 Yrs | 46 | 77 | 97 |
| 50 Yrs | 37 | 70 | 91 |
| 100 Yrs | 35 | 66 | 90 |

| FOR LENGTH OF RECORD = 25 years | | | |
|---|---|---|---|
| | Percent Error Allowed | | |
| Tr | 10% | 25% | 50% |
| 2 Yrs | 68 | 99 | 100 |
| 5 Yrs | 60 | 99 | 99 |
| 10 Yrs | 58 | 95 | 99 |
| 25 Yrs | 50 | 93 | 99 |
| 50 Yrs | 46 | 91 | 97 |
| 100 Yrs | 45 | 89 | 98 |

Table 53. Approximate Values for the Reliability of Estimates of Peak Discharge for Various Lengths of Record and Return Periods (Continued)

| FOR LENGTH OF RECORD = 50 years | | | |
|---|---|---|---|
| | Percent Error Allowed | | |
| Tr | 10% | 25% | 50% |
| 2 Yrs | 87 | 100 | 100 |
| 5 Yrs | 75 | 100 | 100 |
| 10 Yrs | 68 | 96 | 100 |
| 25 Yrs | 58 | 92 | 100 |
| 50 Yrs | 54 | 90 | 100 |
| 100 Yrs | 52 | 90 | 100 |

| FOR LENGTH OF RECORD = 100 years | | | |
|---|---|---|---|
| | Percent Error Allowed | | |
| Tr | 10% | 25% | 50% |
| 2 Yrs | 96 | 100 | 100 |
| 5 Yrs | 91 | 100 | 100 |
| 10 Yrs | 85 | 100 | 100 |
| 25 Yrs | 79 | 100 | 100 |
| 50 Yrs | 73 | 99 | 100 |
| 100 Yrs | 64 | 99 | 100 |

Example: How reliable is an estimate of the $Q_{50}$ peak discharge based upon 25 years of record?

From Table 53: There is a 97 percent chance that the estimate is within $\pm50$ percent of the correct value, a 91 percent chance that the estimate is within $\pm25$ percent of the correct value and only a 46 percent chance that the estimate is within $\pm10$ percent of the correct value.

From Table 53 it is clear that the estimates for peak discharges with recurrence intervals of 50 years or more can very likely be as much as 25 percent in error, or more. The consequences of the design discharge being higher or lower than the estimated value must be evaluated. The designer then selects a design discharge which provides the optimum balance between all the factors involved.

## 9.3 Least Total Expected Cost

In 1981, Corry et al. prepared the Federal Highway Administration's HEC-17 entitled "Design of Encroachments on Flood Plains using Risk Analysis". This manual contains an in-depth discussion of the least total expected cost (LTEC) design process and many illustrative examples for computing economic losses and the LTEC design analysis.

Whenever a highway encroaches on a flood plain an evaluation of the related risks to the highway facility and to the surrounding property is advisable. When the early evaluation indicates that a reasonable expectation of risk exists, a detailed analysis of alternative designs is necessary in order to determine the design with the least total expected cost (LTEC) to the public.

Risk analysis is basic to the LTEC method and permits the analysis of economic losses associated with flooding probabilities for various design options. All quantifiable losses are included in a risk analysis. These may involve damage to structures, embankments, surrounding property, traffic related losses, and scour or stream channel damage. The sum of the annual economic risk cost, the annual capital costs, and the total construction costs multiplied by a capital recovery factor, results in the total expected cost (TEC) for each design option. Comparison of the various TEC's for all design strategies allows the designer to select the LTEC or optimum design strategy.

The determination of whether or not to design by the LTEC process is a screening process. All encroachments should be assessed against engineering established criteria consisting of the following: 1) lack of a practicable detour, 2) substantial hazard to people, and 3) substantial hazard to property. If any of the criteria is exceeded, the encroachment should be designed by the LTEC process.

To illustrate the principles of the LTEC method, the following simple example is taken directly from HEC-17. In this example, it is assumed that the economic losses have been previously assessed using methods of HEC-17 and are given as input data to the example.

Example:

It is desired to design a circular culvert under a two-lane highway. The culvert length is 100 feet. The equivalent average daily traffic is 3000 vehicles per day. The discount rate used is 7-1/8 percent and the useful life of the structure is 35 years.

The flood range used in the analysis is:

| Return Period | Exceedance Probability | Discharge (CFS) |
|---|---|---|
| 5 | 0.02 | 100 |
| 10 | 0.10 | 150 |
| 20 | 0.05 | 170 |
| 40 | 0.025 | 190 |
| 80 | 0.0125 | 200 |
| 160 | 0.00625 | 230 |

The alternative designs included are:

| Culvert Diameter (in) | Elev. Top of Fill (ft) |
|---|---|
| 48 | 316 |
| 54 | 316 |
| 60 | 316 |
| 66 | 316 |

The economic losses due to traffic interruption, backwater and damage to the embankment have been assessed, the results of which are given below.

Economic Losses

| Culvert Diameter (in) | Fill Elev. (ft) | Exceedance Probability 0.20 | 0.10 | 0.05 | 0.025 | 0.0125 | 0.00625 |
|---|---|---|---|---|---|---|---|
| 48 | 316 | 0 | 150 | 375 | 490 | 650 | 928 |
| 54 | 316 | | 0 | 105 | 275 | 460 | 710 |
| 60 | 316 | | | | 0 | 159 | 510 |
| 66 | 316 | | | | | 0 | 248 |

The annual capital and maintenance costs are:

| Culvert Diameter (in) | Capital Cost ($) | Annual Capital Cost ($) | Annual Maintenance Cost ($) | Annual Culvert Cost ($) |
|---|---|---|---|---|
| 48 | 4090 | 355 | 25 | 380 |
| 54 | 5340 | 463 | 20 | 483 |
| 60 | 6600 | 573 | 15 | 588 |
| 66 | 8320 | 722 | 10 | 732 |

The annual risk costs are best computed in tabular form as shown below for the 48-inch diameter culvert. The probabilities and economic losses are obtained from the above tables for flood ranges and economic losses, respectively. The average economic losses are then computed for incremental probabilities or the number of exceedances within a probability range. The incremental probable annual damages or annual risk is the product of the incremental probabilities and the average losses for each flow increment. The total annual risk is the sum of the incremental annual risks.

The annual risk costs for the 48-inch culvert are:

| Q (CFS) | Probability | Losses ($) | Average Losses ($) | Delta Probability | Annual Risk ($) |
|---|---|---|---|---|---|
| 100 | 0.20 | 0 | | | |
| | | | 75.00 | 0.10 | 7.50 |
| 150 | 0.10 | 150 | | | |
| | | | 262.50 | 0.05 | 13.13 |
| 170 | 0.05 | 375 | | | |
| | | | 432.50 | 0.025 | 10.81 |
| 190 | 0.025 | 490 | | | |
| | | | 570.00 | 0.0125 | 7.13 |
| 200 | 0.0125 | 650 | | | |
| | | | 789.00 | 0.00625 | 4.93 |
| 230 | 0.00625 | 928 | | | |
| | | | 928.00 | 0.00625 | 5.80 |
| | 0 | 928 | | | |

Risk = 7.50 + 13.13 + 10.81 + 7.13 + 4.93 + 5.80

Risk = $49.30

The total expected cost for the 48-inch diameter culvert is then the sum of the total annual risk and the annual capital cost.

The annual risk costs for all the other alternative designs (culvert sizes) are computed in an analogous manner and combined with the annual capital cost as tabulated in the total expected cost (TEC) table below.

| Culvert Diameter (in) | Annual Capital Cost ($) | Annual Risk Cost ($) | Total Expected Cost ($) |
|---|---|---|---|
| 48 | 380 | 49.30 | 429.30 |
| 54 | 483 | 20.07 | 503.07 |
| 60 | 588 | 6.28 | 594.28 |
| 66 | 732 | 2.32 | 734.32 |

The LTEC design is therefore the 48-inch culvert. Figure 83 shows a comparison of the annual cost of the alternative designs.

In the above example, it was assumed that the culvert did not fail under any of the flood conditions. If the culvert is assumed to fail when the embankment losses are greater than 50 percent the following results are obtained. The culvert failure is treated as an additional loss by adding the cost to replace (using initial cost data) in the computation of the annual risk costs. The failure criteria is triggered only for the 48-inch culvert design for floods of 190 CFS or greater. The computations for the annual risk for the 48-inch culvert are again shown below.

| Q (CFS) | Probability | Losses ($) | Average Losses ($) | Delta Probability | Annual Risk ($) |
|---|---|---|---|---|---|
| 100 | 0.20 | 0 | | | |
| | | | 75.00 | 0.10 | 7.50 |
| 150 | 0.10 | 150 | | | |
| | | | 262.50 | 0.05 | 13.13 |
| 170 | 0.05 | 375 | | | |
| | | | 2477.50 | 0.025 | 61.93 |
| 190 | 0.025 | 4580 | | | |
| | | | 4660.00 | 0.0125 | 58.25 |
| 200 | 0.0125 | 4740 | | | |
| | | | 4879.00 | 0.00625 | 30.49 |
| 230 | 0.00625 | 5018 | | | |
| | | | 5018.00 | 0.00625 | 31.36 |
| | 0.0 | 5018 | | | |

Risk = 7.50 + 13.13 + 61.93 + 58.25 + 30.49 + 31.36

Risk = $202.66

The total expected cost for each design option is recomputed as tabulated below.

| Culvert Diameter (in) | Annual Capital Cost ($) | Annual Risk Cost ($) | Total Expected Cost ($) |
|---|---|---|---|
| 48 | 380 | 202.66 | 582.66 |
| 54 | 483 | 20.07 | 503.07 |
| 60 | 588 | 6.28 | 594.28 |
| 66 | 732 | 2.32 | 734.32 |

In this case the LTEC design changes to the 54-inch culvert as illustrated in Figure 84.

The overall objective is to determine an alternative which provides the greatest protection for the Least Total Expected Cost (LTEC). Admittedly, this compromise of cost versus protection is a difficult one to arrive at in many cases. However, the LTEC method discussed above is one such procedure which has as its goal to minimize costs which are made up of the initial cost, maintenance charges, and the cost of any damage which results from the insufficiency of the structure. The designer is encouraged to utilize this procedure to aid in the selection of a final design. It is relatively simple, readily lends itself to automation and can be easily and quickly updated with cost data on an annual or other selected basis.

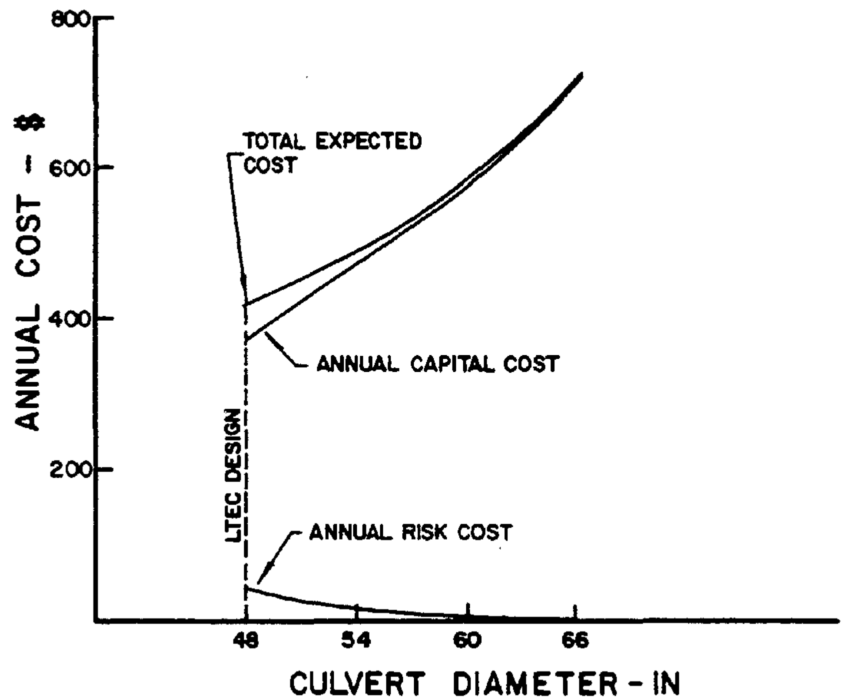



Figure 83. Least Total Expected Cost - Culvert Design Without Failure

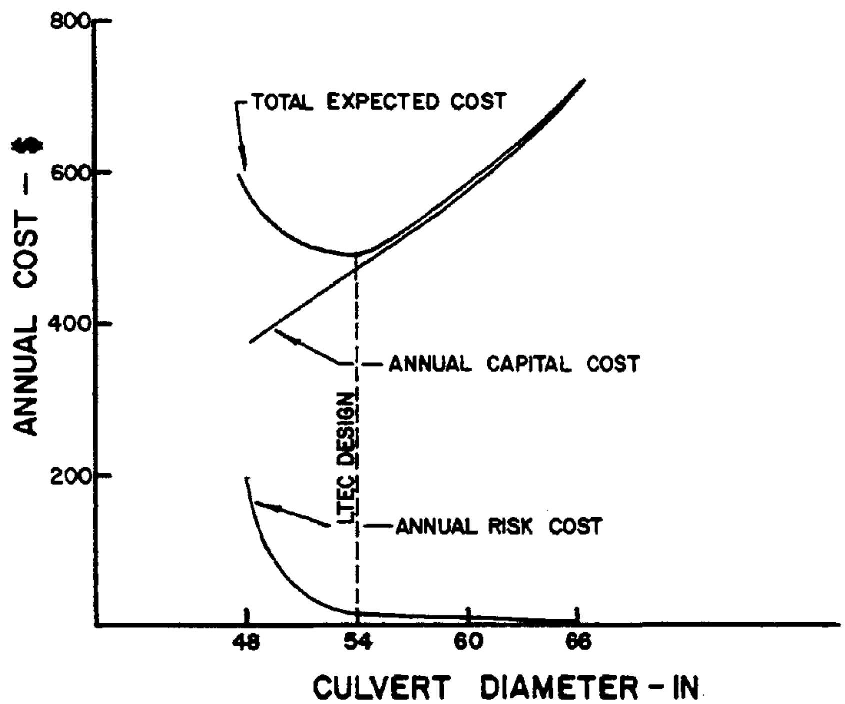



Figure 84. Least Total Expected Cost - Culvert Design With Failure

### 9.4 Probable Maximum Flood

On occasion, hydraulic structures are constructed where a failure would be catastrophic. The potential for loss of life, disruption of essential services and excessive economic damages require a structure to be safe at a design discharge equal to the Probable Maximum Flood (PMF). For a particular basin, the PMF is the flood which results from a hypothetical storm defined as the Probable Maximum Storm (PMS).

The development of the PMF is basically a three step process. The first step is to determine the Probable Maximum Precipitation (PMP). The PMP is defined as the greatest depth of rainfall of a given duration that is physically possible in a particular geographical area. It is determined from hydrometeorological studies involving the maximization of the possible moisture in the atmosphere, transposition of storms to the area of interest and envelopment of the maximum precipitations for various durations and areas for the purpose of data fill-in. Such meteorological studies are very detailed and require a great amount of effort. The U.S. Weather Bureau, 1978, has prepared generalized charts giving PMP estimates in the United States east of the 105th meridian for specified durations of 6 to 72 hours and areas of 10 to 20,000 square miles. The estimates are all-season and therefore represent the greatest amounts of precipitation for any time of the year. A similar report by the U.S. Weather Bureau, 1983, (in draft form) gives PMP estimates for the United States between the Continental Divide and the 103rd meridian.

With the PMP determined, the Probable Maximum Storm (PMS) is then configured taking into account the spatial distribution of the PMP as governed by shape, orientation, movement, and storm-area size, and the temporal distribution of the precipitation during the storm. The Corps of Engineers, 1984, describe in detail the determination of the PMP and PMS and discuss the computer program HMR52 to facilitate these computations.

After the PMS is developed, the probable maximum flood (PMF) is determined by the various hydrograph methods discussed in Sec. 5.0 of this manual. The U.S. Bureau of Reclamation, 1961, presents a very extensive discussion of the PMF and illustrates the development of the PMF hydrograph by a detailed example using the SCS triangular unit hydrograph method.

### 9.5 Importance of Hydrology to Risk Analysis

In HEC-17, Corry et al. clearly point out the differences in design by traditional concepts and by risk analysis. In the case of traditional design, the peak flow at a predetermined frequency of occurrence is normally the single most important input design parameter. Structures are sized to handle this design flow. There is still an element of risk due to the probabilistic nature of the flooding in this design approach. However, the risk is only implicit in the design standards of a pre-selected frequency flood and in the limitations that may be placed on stage, backwater, velocities and other factors determinable from the design flood.

With risk analysis, the design discharge ceases to be an input parameter. Instead, a range of discharges is used in the analysis, and the selected design discharge results from the analysis which yields a least total expected cost for the design project. Risk is explicitly defined and quantified in the analysis for all reasonable design options. The traditional concept of a design discharge is well entrenched in highway design as it is in other fields requiring the design of hydraulic structures. This was especially evident in Table 31 where among the State Highway projects surveyed in ungaged watersheds, 95 percent involved only peak flow determination from either state regression equations, other empirical formulas or extrapolation from gaged sites. Although it is recognized that there is considerable inertia to be overcome in changing from traditional design practice, it is becoming increasingly more important that drainage design be cost effective and commensurate with the potential risk. This is especially true in light of the large fraction of highway construction dollars spent on drainage structures and the increasing number of bridges, culverts and other hydraulic appurtenances due for replacement or rehabilitation.

In the previous sections of this manual, considerable emphasis has been placed on methods for flood frequency analysis and the development of flood hydrographs for both urban and nonurban watersheds. Aside from its purpose as an instructional guideline for carrying out the various analytical procedures, the manual has also provided the basic computational methods to determine the hydrologic inputs for application of risk analysis, damage evaluation and the least total expected cost method of design.

APPENDIX A

LIST OF REFERENCES

American Society of Civil Engineers, "Design Manual for Storm Drainage", New York, 1960.

Beard, Leo R., "Statistical Methods in Hydrology", U.S. Army Engineer District, Corps of Engineers, Sacramento, California, 1962.

Benson, M.A., "Evolution of Methods for Evaluating the Occurrence of Floods", U.S. Geological Survey Water Supply Paper 1580-A, 1962.

Corry, M.L., Jones, J.S. and Thompson, P.L., "Design of Encroachments of Flood Plains Using Risk Analysis", HEC-17, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1981.

Dalrymple, Tate, "Flood Frequency Analyses", U.S. Geological Survey, Water Supply Paper 1543-A, 1960.

Dawdy, D.R., Lichty, R.W. and Bergmann, J.M., "A Rainfall-Runoff Model for Estimation of Flood Peaks for Small Drainage Basins", U.S. Geological Survey Professional Paper No. 506-B, 1972.

Fletcher, J.E., Huber, A.L., Haws, F.W. and Clyde, C.G., "Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method", Volume I, Research Report No. FHWA-RD-77-159 and Volume II, Recommendations for Preparing Design Manuals and Appendices B, C, D, E, F, G and H, Rep. No. FHWA-RD-77-160, Federal Highway Administration, Offices of Research & Development, Washington D.C., October, 1977.

Hampton, B.B. and Wood, C.M., "Hydrologic Data for Urban Studies in the Dallas, Texas Metropolitan Area", U.S. Geological Survey, Open File Report No. 77-381, June, 1977.

Hampton, B.B. and Wood, C.M., "Hydrologic Data for Urban Studies in the Dallas, Texas Metropolitan Area", U.S. Geological Survey, Open File Report No. 80-1004, September, 1980.

Huff, F.A., "Time Distribution of Rainfall in Heavy Storms", Water Resources Research, Vol 3, No. 4, 1967.

Jackson, N.M., "Magnitude and Frequency of Floods in North Carolina", U.S. Geological Survey, Water Resources Investigations, 76-17, 1976.

Kite, G.W., "Frequency and Risk Analyses in Hydrology", Water Resources Publications, Fort Collins, Colorado, 1977.

LIST OF REFERENCES
(Continued)

Langbein, W.A., "Annual Floods and the Partial Duration Flood Series", Trans., American Geophysical Union, Vol 30, Dec., 1949.

Liscum, Fred and Massey, B.C., "Technique for Estimating the Magnitude and Frequency of Floods in the Houston, Texas, Metropolitan Area", U.S. Geological Survey, Water Resources Investigations 80-17, Austin, Texas, 1980.

Morel-Seytoux, H.J. and Verdin, J.P., "Extension of the Soil Conservation Service Rainfall-Runoff Methodology for Ungaged Watersheds", Report No. FHWA-RD-81-060, Federal Highway Administration, Offices of Research & Development, Environmental Division, Washington D.C., July, 1981.

Newton, D.W. and Herrin, J.C., "Assessments of Commonly Used Methods of Estimating Flood Frequency", Transportation Research Record 896, Transportation Research Board, National Research Council, National Academy of Sciences, Washington D.C., 1982.

Potter, W.D., "Peak Rates of Runoff from Small Watersheds", Bureau of Public Roads, Hydraulic Design Series, No. 2, 1961.

Riggs, H.C., "Some Statistical Tools in Hydrology", Techniques of Water Resources Investigations of the United States, Geological Survey, Book 4, Chapter A1, 1968.

Sanders, Thomas G. (Editor), "Hydrology for Transportation Engineers", U.S. Department of Transportation, Federal Highway Administration, 1980.

Sauer, V.B., Thomas, W.O., Stricker, V.A. and Wilson, K.V., "Flood Characteristics of Urban Watersheds in the United States", U.S. Geological Survey, Water Supply Paper 2207, Washington, D.C., 1983.

Schroeder, E.E. and Massey, B.C., "Technique for Estimating the Magnitude and Frequency of Floods in Texas", U.S. Geological Survey, Water Resources Investigations 77-110, 1977.

Snyder, F.M., "Synthetic Unit Graphs", Trans., American Geophysical Union, Vol. 19, 1938.

Soil Conservation Service, "Method for Estimating Volume and Rate of Runoff in Small Watersheds", TP 149, U.S. Department of Agriculture, Soil Conservation Service, Washington D.C., revised April, 1973.

APPENDIX A

LIST OF REFERENCES
(Continued)


Soil Conservation Service, "Soil Conservation Service National Engineering Handbook", Section 4, HYDROLOGY, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 1972.

Soil Conservation Service, Technical Release No. 20, U.S. Department of Agriculture, Soil Conservation Service, 1965.

Soil Conservation Service, "Urban Hydrology for Small Watersheds", Technical Release No. 55, U.S. Department of Agriculture, Soil Conservation Service, 1975.

Stricker, V.A. and Sauer, V.B., "Techniques for Estimating Flood Hydrographs for Ungaged Urban Watersheds", U.S. Geological Survey, Open File Report 82-365, April, 1982.

Texas Department of Highways and Public Transportation, Hydraulics Manual, Texas Department of Highways and Public Transportation, 1970.

U.S. Army Corps of Engineers, "HEC-1 Flood Hydrograph Package, Users Manual", Hydrologic Engineering Center, January, 1973.

U.S. Army Corps of Engineers, "HMR52 - Probable Maximum Storm (Eastern United States) User's Manual", Water Resources Support Center, 1984.

U.S. Bureau of Reclamation, "Design of Small Dams", U.S. Government Printing Office, Denver Colorado, 1961.

U.S. Department of Commerce, NOAA and U.S. Army, Corps of Engineers, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", Hydrometeorological Report No. 51, Washington D.C., 1978.

U.S. Department of Commerce, NOAA, "Probable Maximum Precipitation Estimates, United States between the Continental Divide and the 103rd Meridian", Hydrometeorological Report No. 55 (draft), Washington, D.C., 1983.

U.S. Geological Survey, "Codes for the Identification of Hydrologic Units in the United States and the Caribbean Outlying Areas", U.S. Geological Survey Circular 878A.

U.S. Geological Survey, Surface Water Branch, Technical Memorandum 73-16, U.S. Geological Survey, 1973.

APPENDIX A

LIST OF REFERENCES
(Continued)


U.S. Weather Bureau, "Maximum Recorded United States Point Rainfall", Technical Paper No. 2, U.S. Department of Commerce, Washington D.C., 1947.

U.S. Weather Bureau, "Rainfall Frequency Atlas of the United States", Technical Paper No. 40, U.S. Department of Commerce, Washington, D.C., 1961.

Verdin, J.P. and Morel-Seytoux, H.J., "User's Manual for XSRAIN - A FORTRAN IV Program for Calculation of Flood Hydrographs for Ungaged Watersheds", Report No. FHWA-RD-81-061, Federal Highway Administration, Offices of Research & Development, Environmental Division, Washington D.C., July 1981.

Water Resources Council, Hydrology Committee, "Estimating Peak Flow Frequencies for Natural Ungaged Watersheds - A Proposed Nationwide Test", U.S. Water Resources Council, Washington, D.C., 1981.

Water Resources Council, Hydrology Committee, "Guidelines for Determining Flood Frequency", Bulletin 17B, (Revised) U.S. Water Resources Council, Washington, D.C., Revised, Sept., 1981.

Yen, B.C. and Chow, V.T., "Local Design Storm", Volume I-IV, Report Nos. FHWA-RD-82-063 to 066, Federal Highway Administration, Office of Research & Development, Washington D.C., May, 1983.

APPENDIX B

# GUIDELINES
# FOR THE EVALUATION OF HIGHWAY ENCROACHMENTS
# ON FLOOD PLAINS

Hydraulics Branch

Bridge Division

Office of Engineering

Federal Highway Administration

February 1982

CONTENTS

GUIDELINES FOR THE EVALUATION OF ENCROACHMENTS ON FLOOD PLAINS

The following outline, along with sources (Appendix A), presents an approach to the evaluation of highway encroachments on flood plains. This approach, when implemented by drainage design and highway location specialists, should satisfy the requirements of Executive Order 11988, "Floodplain Management," DOT Order 5650.2 "Floodplain Management and Protection," and FHPM 6-7-3-2, "Location and Hydraulic Design of Encroachments on Flood Plains." The decisionmaking process established by FHPM 6-7-3-2, which is the basis of these guidelines, is illustrated in Appendix B.

1. Location Hydraulic Studies (1)[a] [7][b]

    a. Office Review (A checklist similar to Appendix C is useful)
        (1) Collect data (8)
            (a) Locations of highway alternatives on a site map (USGS 7 1/2 min. quad sheets, aerial photos, highway location mapping (1" = 200'), State and county highway maps)
            (b) Available hydraulic and hydrologic information
                1 Previous highway drainage studies
                2 National Flood Insurance Program (NFIP) maps and studies (2) [7a]
                3 Other flood data
                    a USGS (Water Supply Papers, State reports, etc.)
                    b High water marks, etc.
                    c WATSTORE (USGS)
                4 Planning studies of water resource agencies (3)
                    a Corps of Engineers
                    b Local conservancy districts, drainage districts, etc.
                    c River Basin Commissions
                    d Coastal Zone Management Agencies
                    e Soil Conservation Service
                    f Bureau of Land Management
                5 Location of water courses and determination of drainage areas - USGS Quad Sheets, 1/250,000 maps, aerial photos, etc.
            (c) Present and future land use and culture in the transportation corridor (USGS 7 1/2 min. quad sheets, local and regional planning reports, aerial photographs)
        (2) Make preliminary estimates and studies
            (a) Make preliminary hydrologic estimates at probable encroachment sites
            (b) Estimate flood limits where necessary to determine encroachments
            (c) Make any preliminary hydraulic studies necessary to assess significance of encroachment

a - Underlined numbers in parenthesis indicate reference citations found in Appendix A.
b - Numbers in brackets indicate the appropriate paragraph of FHPM 6-7-3-2, dated November 15, 1979.

(3) Identify probable encroachments on base flood plains [7a]
    (a) Prepare a list of probable encroachments and associated potential risks [4o], impacts [4i], and supports [4r]
    (b) Select for field review encroachments which:
        1 could be significant [4q] or longitudinal
        2 could require a preliminary hydraulic study [9a]
        3 could have potential problems with support of incompatible flood plain development

b. Field Review of Selected Encroachments
   (1) Determine by visual observation the likelihood of the encroachment [7a] and verify data (flood plain limits, etc.) collected prior to the field trip.
   (2) For crossings - Consider the desirability of the encroachment location alternative from a hydraulic viewpoint (Is the crossing located at the right point in the river: skew, auxiliary waterway openings, local drainage, confluences, bends etc.) (1)
   (3) For longitudinal encroachments - Is an alternative location which does not encroach on the base flood plain practicable? [4K] (Consider the effects on topography and culture; e.g., large cuts, intrusion into neighborhoods, additional costs, etc.) [7b]
   (4) For probable encroachments, investigate potential impacts and mitigation measures. [7c]
       (a) Risk [4o]
           1 Existing - Verify the data collected prior to the field trip regarding existing development. Decide whether flooding problems are likely to exist and whether the proposed highway facility will impact adversely on the existing situation.
           2 Impacts - Effect on land use and development within flood plain limits, channel stability, bank stability, bends and meanders, aggradation, degradation, necessity for channel change, debris and ice, skew of crossing.
           3 Measures to minimize potential impacts. (3)
       (b) Natural and beneficial flood plain values [4i]
           1 Impacts - Effects on the environment, fish and other wildlife, water supplies, recreational resources, etc.
           2 Measures to minimize impacts. (3)(15)(20)
           3 Measures to restore and preserve the function of those values which are adversely affected. (3)
       (c) Probable support of incompatible flood plain development and measures to minimize impacts, risks and supports.
       (d) Potential for interruption or termination of a transportation facility which is needed for emergency vehicles or provides a community's only evacuation route.
   (5) For probable significant encroachments - Can the significant impact be avoided in a practicable manner by shifting the alignment or modifying the design?

c. After the Field Review

(1) Delineate base flood plain limits as necessary to identify encroachments and impacts.

   (a) Use NFIP maps (These maps usually only indicate base flood plains that are wider than 100 feet.)

      1 A Flood Insurance Rate Map (FIRM) or Flood Insurance Study (FIS) report should be referred to first.

      2 If a FIRM or FIS is not available, a Flood Hazard Boundary Map (FHBM) should be used to determine if an alternative clearly does include an encroachment.

      3 If a detailed study indicates that a FIRM is inaccurate, flood-plain limits may be appealed using FEMA procedures in 44 CFR 68.

   (b) Obtain maps or calculations of others (43 FR 6049) (3)

   (c) Determine by analytic means (degree of refinement needed to be determined on a case by case basis, commensurate with the risk involved).

(2) Identify encroachments where avoidance is practicable [4k] and make corresponding changes to the alignment(s); document any additional costs, tradeoffs and other impacts required to avoid the encroachments. [7b&d]
(This will require coordination with other disciplines, e.g., geometric, safety, and geotechnical specialists.)

(3) Identify and list encroachments that apparently cannot be avoided. Consider localized line shifts to avoid or minimize the impacts of these encroachments. [7b&d]

(4) Evaluate potential support of any incompatible flood plain development that is likely to occur as a result of the project. [7c&d]

(5) Determine consistency with regulatory floodways. [9a5]

(6) Coordinate findings with appropriate Federal, State and local water resources/environmental agencies.

Comments: Up to this point, the process envisioned is primarily one of identification and classification of encroachments on the basis of field reconnaissance and analysis of data by highway drainage specialists. It is highly desirable that the appropriate State and FHWA environmental and engineering personnel be directly involved with the location hydraulic studies, including field trip(s) to probable encroachment sites. The understanding of the project gained through field reconnaissance adds immeasurably to the ability of these personnel to make decisions about the project; thus field reconnaissance should not be delegated entirely to consultant personnel or survey crews. Early coordination to obtain the views of the public and water resources/environmental agencies is also important. Normally, the need for actual computations would be expected to be minimal. Encroachments for each location alternative under consideration should be addressed in the development of the draft environmental document. Participation by a drainage specialist will provide for the most cost effective roadway and bridge design and can help to avoid locations that involve conflict.

2. Environmental Review Process (See 23 CFR 771)

a. <u>Draft EIS, Environmental Assessment or Categorical Exclusion</u>

(1) Review issues raised through public involvement procedures. For projects being processed as a categorical exclusion, document results of any location studies, public involvement, etc., in the project records. [7e]

(2) Present results of studies in draft environmental review document.

(a) Include an exhibit which displays both the alternatives and the approximate 100-year flood plain, as appropriate. [7a]

(b) Summarize the results of location hydraulic studies for each alternative. [7e]

(c) Indicate consistency with existing or proposed regulatory floodways and appropriate coordination. [9a5]

(d) Discuss practicability of alternatives to significant encroachments. [7d]

(3) Through public involvement processes, advise public of the on-going flood plain studies.

b. <u>Final EIS or FONSI</u>

(1) Review issues raised through public involvement procedures. Reevaluate the alternatives on the basis of the comments received and water resources concerns, including support of any incompatible flood plain development.

(2) After selection of the preferred location alternative for the final environmental document, review the alignment to see if any further efforts can be made to minimize encroachments or their impacts, considering input from the public and review agencies. Review the adequacy of hydrologic and hydraulic studies for assessment purposes, expanding them as necessary.

(3) Prepare responses to comments received. Meet with water resources agencies/public as necessary to attempt to satisfy concerns. Involve FHWA regional office personnel if major concerns continue to exist.

(4) Prepare discussion of flood plain impacts (including "only practicable alternative finding" for signature of Regional Highway Administrator (EIS) or of Division Administrator (FONSI) [8a], if appropriate). Comment on significant encroachments.

(5) Document results of the preliminary hydraulic location studies and any commitments made in the environmental process. Make this information available to designers for use in further project development.

(6) Make "only practicable alternative finding" available to State and area-wide clearinghouses. (Suggest sending the final environmental document containing the finding to appropriate clearinghouses.) [8b]

3. Design Hydraulic Studies

a. Office Review

(1) Review checklist (Appendix C) and complete data file initiated in step 1a(1)
   (a) Obtain alignment and profile of selected alternative
   (b) Update list of encroachments and associated assessment
   (c) Obtain commitments made in environmental documents, step 2
   (d) Review drainage areas
      1 Check area determined in step 1a(1)(b)5
      2 Determine areas of additional encroachments
   (e) Refer to flood hazard studies for area (2) and review flood plain zoning

(2) Hydrologic analysis (5)(6)
   (a) Make final hydrologic estimates
   (b) For selected encroachments (bridges and others as appropriate)
      1 List available flood-frequency records and flood studies, etc.
      2 Evaluate potential for changes in watershed characteristics which would change magnitude of flood peaks; e.g., urbanization, channelization
      3 Plot flood-frequency curve
      4 Determine distribution of flow and velocities for several discharges or stages in natural channel for existing conditions
      5 Plot stage-discharge-frequency curve

(3) Site map - used for estimating flood flow distribution, selecting cross sections of stream, showing locations of proposed encroachment and structure(s), and indicating existing features (stream controls, encroachments, development, and highway structures)
   (a) Select type
      1 Specially prepared map showing contours, vegetation and improvements.
      2 In some cases, cross sections normal to floodflow are acceptable in lieu of map. Determine number of sections necessary.
   (b) Prepare instructions for survey party indicating features to map

(4) Survey data - select encroachments to review in the field and initiate survey data report (such as Appendix D) which includes the following:
   (a) Photographs (showing existing structures, past floods, main channel and flood plain) to document existing conditions and to use in assigning resistance values.
   (b) Comments on drift, ice, nature of streambed, bank stability, bend meanders, vegetative cover and land use.
   (c) Factors affecting water stages - highwater from other streams, reservoirs (existing or proposed and approximate date of construction), flood control projects (give status), tides, and other controls.
   (d) Locations and elevations of highwater marks along stream, giving dates of occurrence.

(e) The relative importance and/or value of adjacent property and, where appropriate, a list of facilities susceptible to flooding and first floor elevations.
(f) Features which are constraints to modifying the upstream water surface elevation.
(g) Evaluation of the need for riprap and/or scour protection including the need for spur dikes, energy dissipators, countermeasures, etc.
(h) Location of existing structures (including relief or overflow structures) with respect to proposed crossing or encroachment (upstream, downstream, as well as existing roadway) and describe each fully (Appendix D), giving:
  1 Type, including span lengths and number of spans, bent design, pier orientation, culvert size, number of cells.
  2 Foundation type (spread footing, piling) and depth.
  3 Scour history at abutments, bents, culvert outlets; head cutting; stream aggradation and degradation.
  4 Cross section beneath structures, noting clearance to superstructure and skew with direction of current during extreme floods (add to survey party instructions).
  5 Flood history, highwater marks (dates and elevation), nature of flooding (including overtopping), damages and sources of information.
  6 Damage from abrasion, corrosion, wingwall failure, culvert end failure.

b. Field Review (The drainage specialist designing the project should review all locations that will require drainage structures. Where appropriate, this review should be combined with the location field review.)
  (1) Collect information for a final assessment of the risks, impacts, and supports and measures to minimize, restore, and preserve that were determined in step 1b(4).
  (2) Review survey data collected in step 3a(4).

c. Hydraulic Analysis (7)

  (1) Review field report (Appendix D) and update data file and checklist (Appendix C).
  (2) Using the assessment of each encroachment, determine the appropriate method for studying design alternatives: mathematical model, physical model or both.
  (3) Rate capacity of existing features located in steps 3a(4)(c) and (h) and if necessary adjust the stage-discharge-frequency relationship estimated in step 3a(2)(b)5.

(4) Design of bridge waterways (4)(8)(9)(19)

- (a) Identify features which are constraints to modifying the upstream water surface elevation:
  - 1 Land use
  - 2 Development
  - 3 Watershed divides
  - 4 Flood plain values, e.g. wetlands, etc.
- (b) Determine navigation requirements and evaluate need for channel modifications and controls
- (c) Compute backwater for various bridge lengths, approach profiles and discharges
  - 1 Review flow distribution determined in step 3a(2)(b)4 and consider need for auxiliary structures
  - 2 Plot data as a family of curves on the stage-discharge-frequency curve developed in step 3a(2)(b)5 for existing conditions. (4)(10)
- (d) Select encroachment design [9a(1)]:
  - 1 By risk analysis (9) or
  - 2 By assessment of the risks
- (e) Estimate scour depth at piers and abutments (8)
- (f) Design embankment, bank and channel protection and scour attenuation devices, if required (11)(12)(13)(14)
- (g) Investigate need for and design spur dikes (4)

(5) Design of culverts (15)

- (a) Identify features which are constraints on headwater elevation and highway profile
- (b) Evaluate abrasion and corrosion potential
  - 1 Eliminate from consideration materials that will give unsatisfactory service life or
  - 2 Choose protective measure
- (c) Compute and plot performance curves for trial culvert sizes (16)
- (d) Evaluate need for and provisions for fish passage
- (e) Select culvert design
  - 1 By risk analysis (9) or
  - 2 By assessment of the risks
- (f) Determine hydraulically equivalent sizes for bid alternatives
- (g) Evaluate need for and design for debris control (17)
- (h) Evaluate need for and design for outlet protection (13)
- (i) Investigate need for and design for protection against failure by buoyancy and/or by separation at joints

(6) Design of longitudinal encroachments (18)(19)
   (a) Determine navigation requirements and evaluate need for channel modifications and controls
   (b) Determine the effect of proposed encroachment on water-surface profiles using various roadway profile alternatives
   (c) Select roadway profile design [9a(1)]
      1 By risk analysis (9) or
      2 By assessment of the risks
   (d) Evaluate effects on scour and deposition in channel and tributaries (8)
   (e) Design embankment, bank and channel protection (11)(12)(13)(14)

(7) Documentation
   (a) Show final layout of encroachments in plan and profile, including the magnitude, elevation and exceedance probability of the overtopping flood, the base flood, or, if appropriate, the greatest flood [10c]
   (b) Complete project files, which should include [10b]
      1 Hydrologic and hydraulic data and design computations
      2 Risk assessment or analysis
      3 As appropriate, information on:
         a Navigation requirements
         b Channel modification
         c Effects on stream stability
         d Effects on stream ecology
         e Need for stream controls to protect highway
         f Need and provisions for fish passage

## ATTACHMENT A

## REFERENCES


1. Guidelines for Hydraulic Considerations in Highway Planning and Location, Volume I, Highway Drainage Guidelines, 1973, AASHTO, 341 National Press Building, Washington, D.C. 20045.

2. Contact the appropriate Federal Emergency Management Agency Regional Office or State NFIP coordinator to check if a study is in progress or completed.

3. U.S. Water Resources Council, Floodplain Management Guidelines for Implementing E.O. 11988, 43 FR 6030, February 10, 1978.

4. a. Bradley, J. N., Hydraulics of Bridge Waterways, Hydraulic Design Series No. 1, FHWA, 1970, 111p.

   b. Welty, K. H., Corry, M. L., Morris, J. L., A Computer Program for Hydraulics of Bridge Waterways, Program HY-4-69, FHWA, 1969, 96p.

5. a. Guidelines for Hydrology, Vol. II, Highway Drainage Guidelines, AASHTO, 1973.

   b. FHWA, Hydrology for Transportation Engineers, GPO, January 1980, 736p.

6. a. U.S. Water Resources Council, Bulletin 17A, Guidelines for Determining Flood Flow Frequency, GPO, June 1977.

   b. Flood-frequency analyses, such as those of U.S. Geological Survey or other water resources agencies, for the region in which the structure is located.

7. Guidelines for the Legal Aspects of Highway Drainage, Volume V, Highway Drainage Guidelines, AASHTO, 1977.

8. a. Richardson, E. V., Simons, D. B., Karaki, S., Mahmood, K., Stevens, M. A., Highways in the River Environment, Hydraulic and Environmental Design Considerations, FHWA, May 1975.

   b. Watts, F. J., Addendum to Highways in the River Environment, Hydraulic and Environmental Design Considerations, Hydraulic Engineering Circular No. 16, FHWA, July 1980, 42p.

   c. Keefer, T. N., McQuivey, R. S., Simons, D. B., Stream Channel Degradation and Aggradation: Analysis of Impacts to Highways, FHWA, July 1981.

   d. Shen, H. W., and others, Methods of Assessment of Stream Related Hazards to Highways and Bridges, FHWA/RD-80/160, March 1981.

9. Schneider, V. R., Wilson, K. V., Hydraulic Design of Bridges with Risk Analysis - Example Study and Report, USGS, 1978.

10. a. HEC-2, Water Surface Profiles, Computer Program, U.S. Army Corps of Engineers, 609 Second Street, Davis, California, 95616, (916) 449-2105.

10. b. Shearman, J. O., Computer Applications for Step-Backwater and Floodway Analyses, User's Manual, USGS, Open-file Report 76-499, 1976, 103p.

c. WSP2 Computer Program, A Water Surface Profile Computer Program for Determining Flood Elevations, Technical Release No. 61, SCS, May 1976.

11. Searcy, J. K., Use of Riprap for Bank Protection, Hydraulic Engineering Circular (HEC) No. 11, FHWA, 1967, 43p.

12. Scour at Bridge Waterways, NCHRP Synthesis 5, 1970, Highway Research Board, National Academy of Sciences, 2101 Constitution Avenue, Washington, D.C. 20418

13. Corry, M. L., Thompson, P. L., and others, The Hydraulic Design of Energy Dissipators for Culverts and Channels, HEC No. 14, FHWA, December 1975.

14. Normann, J. M., Design of Stable Channels with Flexible Linings, HEC No. 15, FHWA, October 1975, 136p.

15. Guidelines for the Hydraulic Design of Culverts, Volume IV, Highway Drainage Guidelines, AASHTO, 1975.

16. a. Herr, Lester A., and Bossy, Herbert G., Hydraulic Charts for the Selection of Highway Culverts, HEC No. 5, FHWA, 1965, 54p.

b. Herr, Lester A., and Bossy, Herbert G., Capacity Charts for the Hydraulic Design of Highway Culverts, HEC No. 10, FHWA, 1965, 90p.

c. Harrison, L. J., Morris, J. L., and others, Hydraulic Design of Improved Inlets for Culverts, HEC No. 13, FHWA, 1969, 48p.

d. Marques, M., Electronic Computer Program for Hydraulic Analysis of Culverts (Box and Circular Culverts), HY-6, FHWA, 1979, 164p.

e. Hydraulic Analysis of Pipe-Arch Culverts, Program HY-2, FHWA, 1969, 48p.

f. Wlaschin, P., Hydraulic Design of Improved Inlets for Culverts Using Programable Calculators, CDS#1-Monroe 325, CDS#2-HP 65, CDS#3-TI 59, 1980.

g. Wyoming State Highway Department Hydraulics Section, Culvert Design System, FHWA-TS-80-245, December 1980.

17. Reihsen, G., Harrison, L. J., Debris-Control Structures, HEC No. 9 FHWA, March 1971, 38p.

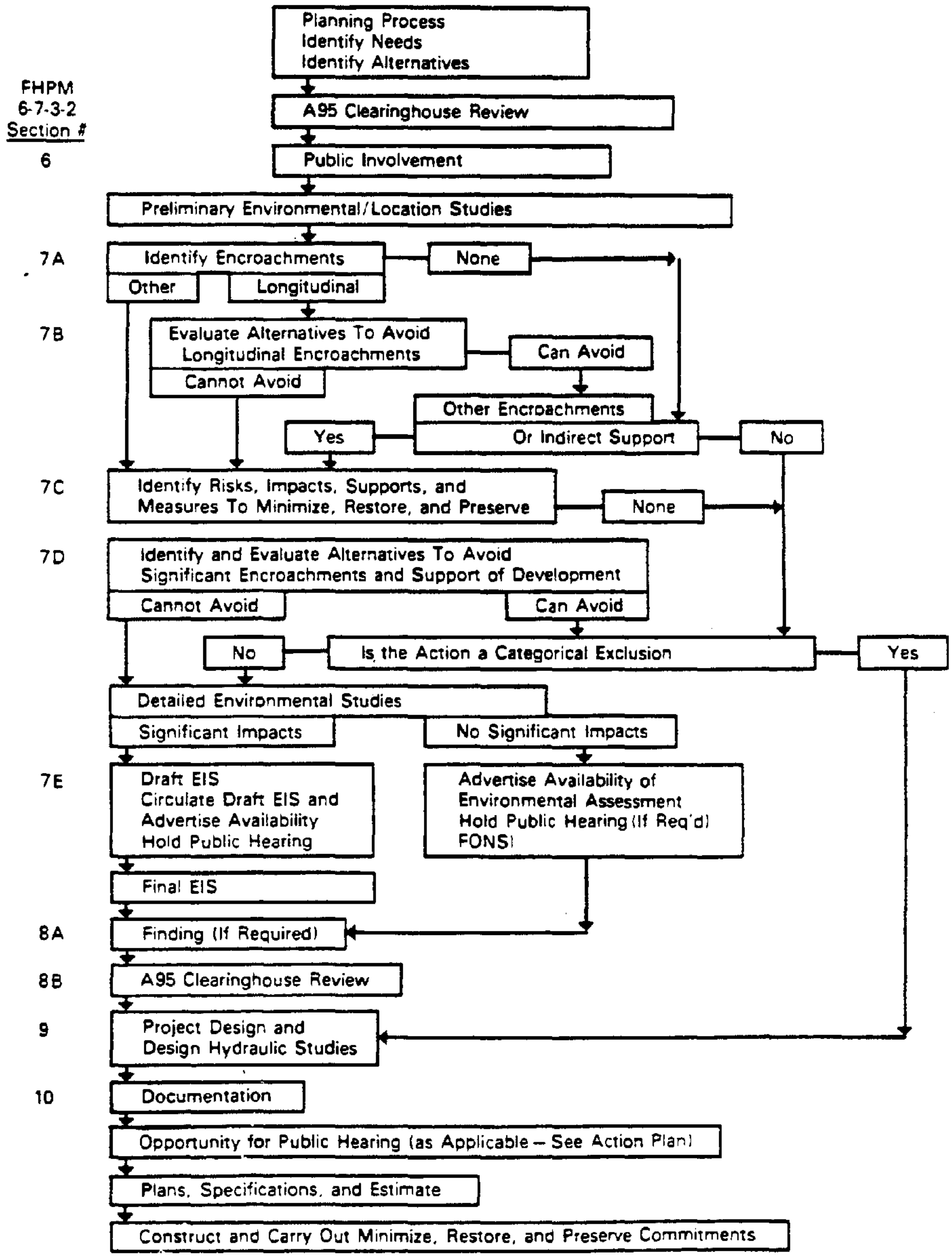
18. Searcy, J. K., Design of Roadside Drainage Channels, Hydraulic Design Series No. 1, FHWA, 1965, 56p.

19. Hydraulic Analysis and Design of Open Channels, Highway Drainage Guidelines, AASHTO, 1979.

20. FHWA Technical Advisory T 5040.12, Corrugated Metal Pipe Durability Guidelines, October 22, 1979.

ATTACHMENT B

# FHWA Flood Plain Management Decisionmaking Process



ATTACHMENT C

L & D-352
4-76

## HYDRAULIC DESIGN PROJECT CHECK LIST

Check Appropriate Items

Rte ________ Proj ________

City County ________ Description ________

**REFERENCE DATA**

| MAPS: | | |
|---|---|---|
| USGS Quad | Scale | Date |
| USGS | 1:250 000 | |
| VDHT | Other | |
| Local Zoning Maps | | |
| Flood Hazard Delineation (Quad.) | | |
| Flood Plain Delineation (HUD) | | |
| | | |
| | | |
| | | |
| Aerial Photos | Scale | Date |
| | | |
| | | |
| | | |

| STUDIES BY EXTERNAL AGENCIES: |
|---|
| USACE Flood Plain Inform. Report |
| SCS Watershed Studies |
| Local Watershed Management |
| USGS Gages & Studies |
| TVA Flood Studies |
| Interim Flood Plain Studies (SWCB) |
| Water Resource Data |
| Regional Planning Data |
| Forestry Service |
| Utility Company Plans |
| |
| |
| |

| STUDIES BY INTERNAL SOURCES: |
|---|
| Quarterly Reports |
| Hydraulics Sect. Records |
| District Drainage Records |
| Flood Records (High Water, Newspaper) |
| |
| |
| |
| |
| |

**HYDROLOGY**

| TECHNICAL RESOURCES: |
|---|
| VDHT Drainage Manual |
| VDHT Directives |
| Technical Library (C.O.) |
| |
| |
| |

| DISCHARGE CALCULATIONS: |
|---|
| Drainage Areas |
| Rational Formula |
| 1021.10 |
| Cir. IV |
| Cir. II |
| Dan Anderson |
| Franklin Snyder |
| Gaging Data - Regional Analysis |
| Regression Equations |
| Area-Discharge Curves |
| Log-Pearson Type III Gage Rating |
| |
| |
| |

| HIGH WATER ELEVATIONS: |
|---|
| VDHT Survey |
| External Sources |
| Personal Reconnaissance |
| |
| |

| FLOOD HISTORY: |
|---|
| External Sources |
| Personal Reconnaissance |
| Maintenance Records |
| |
| |
| |
| |
| |
| |

**HYDRAULIC DESIGN**

| CALIBRATION OF HIGH WATER DATA: |
|---|
| Discharge and Frequency of H.W. el. |
| Influences Responsible for H.W. el. |
| |
| |
| |
| Analyze Hydraulic Performance of Existing Facility for Min. Flow thru 100 Yr |
| Analyze Hydraulic Performance of Prop. Facility for Min. Flow thru 100 Yr |
| |
| |
| |

| DESIGN APPURTENANCES: |
|---|
| Dissipators |
| Rip Rap |
| Erosion & Sediment Control |
| Fish & Wildlife Protection |
| |
| |
| |

| TECHNICAL AIDS: |
|---|
| VDHT Drainage Manual |
| VDHT I & IM'S |
| VDHT & FHWA Directives |
| Technical Library |
| |
| |

| COMPUTER PROGRAMS: |
|---|
| Improved Culvert Inlets |
| Direct Step Water Surface Profile |
| St'd. Step Water Surface Profile |
| USACE HLC-2 Water Surface Profile |
| FHWA Bridge Backwater |
| Log-Pearson Type III Analysis |
| |
| |
| |
| |

| | | | |
|---|---|---|---|
| PRELIMINARY PLAN DEVELOPMENT (IN-DEPTH F.I.) | Assist in Corridor Studies<br>Hydraulic affect on line and grade<br>Potential hydraulic related environmental impact.<br><br>In-Depth Review<br>Review technical check list (reverse side).<br>Estimate drainage requirements that may affect line, grade and project cost.<br>Review project in the field or authorize others to make the review.<br>Obtain additional survey if essential. | R/W STAGE | Obtain necessary additonal survey.<br><br>Answer F. I. questions.<br><br>Finalize drainage design.<br><br>Design appurtenances and obtain spec. design details.<br><br>File Permits. |
| FIELD INSPECTION STAGE | If project is reassigned, check In-Depth file for previous drainage design activity.<br><br>Design all darainage for project except appurtenances and special design items and complete hydrologic data sheet.<br><br>Review technical check list (reverse side).<br><br>Compute or review storm sewer cost participation ratio.<br><br>Review F.I. plans for correct transfer of drainage data.<br><br>Attend F.I., present drainage design and review adequacy of design in field. | CONSTRUCTION STAGE | Review final plans for correct drainage design.<br><br>Final Hydrologic Data Sheet.<br><br>Assist with the solution of problems and revisions during construction. |

ATTACHMENT D
SURVEY DATA REPORT

Project_______________ County_______________
Federal Route Base No. _______ Situation data for design of bridge on Route _______
over_______________
Plane Coordinates or Latitude and Longitude from Highway Department County Map
_______________
Date of Survey:_______________Location (Nearest Town, etc.)_______________

GENERAL INSTRUCTIONS

Fill out all blanks carefully, giving information on all points. High water data is especially important and should be thoroughly investigated. Comments on any item covered in Survey Instruction Manual which are not covered below should be noted on an attached sheet.

## HYDRAULIC SURVEY

### 1. EXISTING STRUCTURE

Existing structure is any structure at, upstream, or downstream from the proposed site have a comparable drainage area.
Date of original construction:_______________
Was present bridge in place at time of extreme high water? _______
Has bridge ever been washed out? _____ Date_______ Mo._______ Yr._______
Explain what portion of bridge or approaches have been washed out:_______________
_______________
Elevation of maximum high water:
Upstream side of existing structure_______________
Downstream side of existing structure_______________
____Ft. upstream of existing structure_______________
____Ft. downstream of existing structure_______________
At other locations on the flood plain (describe)
_______________
_______________
Date of maximum high water: _____ Mo._______ Yr._______ Source of information _______________

### 2. STREAM FLOW DATA AT PROPOSED SITE

Elevation of maximum high water of this stream at proposed location if different from data for existing site:
_____Ft. on upstream side of proposed _______________
_____Ft. on downstream side of proposed_______________
at other locations on the flood plain (describe)_______________
_______________
Date:_____Mo._____ Yr._______ Source of information _______________
_______________
Elevations of highest backwater caused by another stream _______________
Date _______________ Stream Name _______________
Source of information _______________
Elev. of normal water: _________ (Average) Elev. of extreme low water _________
Date:_______________ Mo._________ Year_______________
Source of information _______________
Velocity of current at high water:_____ft./sec. velocity of current at normal water _______________ ft./sec.

### 3. SITE CONDITIONS

Amount and character of drift during a freshet or flood:
_______________

Do banks or bed show scour? ______________________
Description and location of scour: ______________________

______________________

Bed of stream consists mainly of: mud, silt, clay, sand, gravel, cobbles, boulders, soft solid rock, stratified rock, hard rock, silt sedimentation, deposition of large stones, is this material loose or well compacted? ______________________

______________________

Comments on stream ecology and wildlife habitat: ______________________

______________________

## 4. INFLUENCE & CONTROL OF SITE

Location and condition of dams upstream or downstream that will affect high water or discharge at this site: ______________________

______________________

Location and description of any water-gaging stations in the immediate vicinity: ______

______________________

______________________

Elevation ____________ on gage corresponds to elev. ______________________
____________ on survey datum.

Extent to which sink-holes affect runoff, etc.: ______________________

______________________

Brief description of usage of stream for navigational purposes. By small boats, etc. ____

______________________

______________________

______________________

## Railroad Grade Separation Structure Site Data

Railroad milepost ______________________ No. of tracks ______________

Situation data for design of bridge on ______________ over ______________

Type of construction: ______________ New Structure
______________ Replacement of existing structure
______________ Remodeling of existing structure
______________ Paralleling existing structure

Owner of existing structure ______________________.

Owner of grade crossing to be eliminated ______________________.

Date of original construction of any railroad structure being replaced or within approximately 500 feet of the site of a proposed overpass ______________________.

Conditions of existing cut slopes, whether stable, eroded, et cetera ______________
______________. Are ditches open, maintained, et cetera ______________

NOTE - Show cross-section of existing railroad bed at right angles to centerline crossing, with all dimensions, on bridge situation plan. This cross-section should extend from top of cut to toe of fill.

REMARKS -
(Information on significant features not listed, et cetra)

______________________

______________________

______________________

______________________

Survey by ______________________

## APPENDIX C

Federal agencies involved in water related projects which may serve as sources of hydrologic studies, reports or data.

## TABLE OF CONTENTS

Forest Services, U.S. Department of Agriculture

Alaska Region
P.O. Box 1628
Federal Office Building
Juneau, AK 99802

(903) 586-7263
FTS 399-0111

Eastern Region*
633 West Wisconsin Avenue
Milwaukee, WI 53201

(414) 291-3693
FTS 362-3693

Intermountain Region
324 25th Street
Ogden, UT 84401

(801) 625-5605
FTS 586-5605

Pacific Southwest Region
630 Sansome Street
San Francisco, CA 94111

(415) 556-4310
FTS 556-4310

Rocky Mountain Region
11177 W 8th Avenue
P.O. Box 25127
Lakewood, CO 80225

(303) 234-3711
FTS 234-3711

Southeastern Area*
State and Private Forestry
1720 Peachtree Road, NW
Atlanta, GA 30367
(404) 881-4177

Northeastern Area*
State and Private Forestry
370 Reed Street
Broomall, PA 19008

(215) 461-3125
FTS 489-3125

Northern Region
Federal Building
P.O. Box 7669
Missoula, MT 59807

(406) 329-3011
FTS 585-3011

Pacific Northwest Region
P.O. Box 3623
319 SW Pine Street
Portland, OR 97208

(503) 221-3625
FTS 423-3625

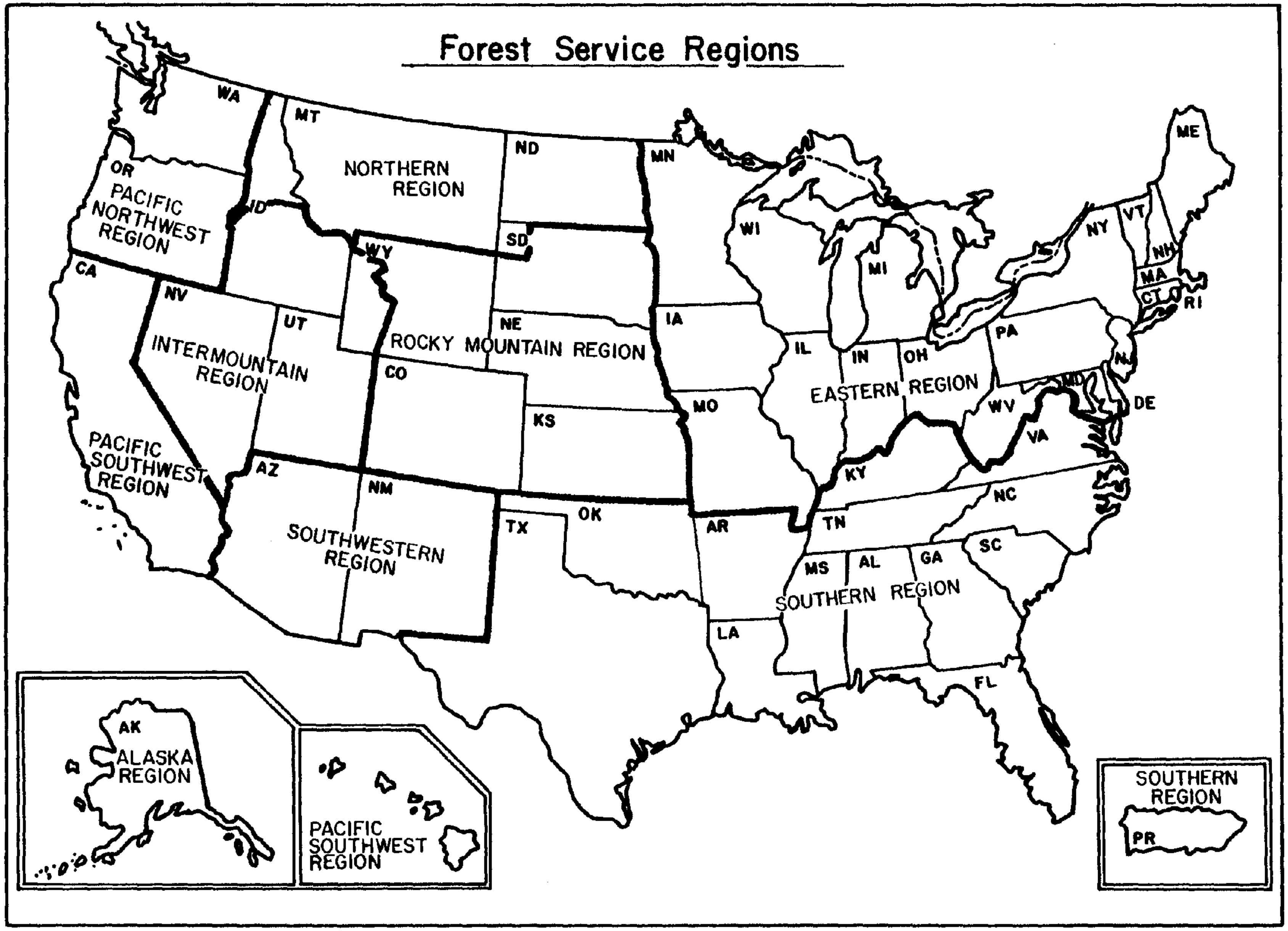
Southern Region*
National Forest System
1720 Peachtree Rd., NW
Atlanta, GA 30367

(404) 881-4177
FTS 257-4177

Southwestern Region
517 Gold Avenue, SW
Albuquerque, NM 87102

(505) 766-2401
FTS 474-2401

* Geographically Coincident


Forest Service Regions
WA
OR
PACIFIC NORTHWEST REGION
MT
ID
NORTHERN REGION
ND
MN
SD
WY
NE
ROCKY MOUNTAIN REGION
CO
KS
CA
NV
UT
INTERMOUNTAIN REGION
PACIFIC SOUTHWEST REGION
AZ
NM
SOUTHWESTERN REGION
TX
OK
WI
MI
IA
IL
IN
OH
PA
NY
VT
NH
ME
MA
CT
RI
NJ
MD
DE
WV
VA
MO
EASTERN REGION
KY
NC
TN
SC
AR
MS
AL
GA
LA
FL
SOUTHERN REGION
AK
ALASKA REGION
PACIFIC SOUTHWEST REGION
SOUTHERN REGION
PR

Soil Conservation Service, U.S. Department of Agriculture

ALABAMA
665 Opelika Rd.
P.O. Box 311
Auburn, AL 36830

(205) 851-8070
FTS 534-4535

ALASKA
2221 E. Northern Lights Blvd.
Suite 129, Prof. Bldg.
Anchorage, AK 99504

(907) 276-4246
FTS 276-4246

ARIZONA
3008 Federal Building
230 N. 1st Avenue
Phoenix, AZ 85025

(602) 261-6711
FTS 261-6711

ARKANSAS
Federal Office Bldg.
700 West Capitol
P.O. Box 2323
Little Rock, AR 72203

(501) 378-5445
FTS 740-5445

CALIFORNIA
2828 Chiles Road
Davis, CA 95616

(916) 449-2848
FTS 449-2848

CARIBBEAN AREA
Federal Office Building
Room 633
GPO Box 4868
San Juan, PR 00936

(809) 753-4206

COLORADO
Diamond Hill Bldg. "A", 3rd Fl.
2490 W. 26th Avenue
P.O. Box 17107
Denver, CO 80217

(303) 837-4275
FTS 437-4275

CONNECTICUT
Mansfield Professional Pk.
Route 44A
Storrs, CT 06268

(203) 429-9361
FTS 244-2547

DELAWARE
Treadway Towers
Suite 210
9 East Loockermen Street
Dover, DE 19901

(302) 678-0750
FTS 487-9148

FLORIDA
Federal Building
401 SE. 1st Avenue
P.O. Box 1208
Gainesville, FL 32602

(904) 377-0946
FTS 377-0946

GEORGIA
Federal Building
355 East Hancock Avenue
P.O. Box 832
Athens, GA 30613

(404) 546-2273
FTS 250-2273

HAWAII
300 Ala Moana Boulevard
P.O. Box 50004
Honolulu, HI 96850

(808) 546-3165
FTS 546-3165

IDAHO
Room 345
304 North 8th Street, Rm. 345
Boise, ID 83702

(208) 334-1601
FTS 554-1601

ILLINOIS
Federal Building
301 North Randolph St.
P.O. Box 678
Champaign, IL 61820

(217) 398-5267
FTS 958-5267

INDIANA
Corporate Square-West
Suite 2200
5610 Crawfordsville Road
Indianapolis, IN 46224

(317) 248-4350
FTS 331-4350

IOWA
693 Federal Building
210 Walnut Street
Des Moines, IA 50309

(515) 284-4260
FTS 862-4260

KANSAS
P.O. Box 600
760 South Broadway
Salina, KS 67401

(913) 823-4565
FTS 823-4565

KENTUCKY
333 Waller Avenue, Rm 305
Lexington, KY 40504

(606) 233-2749
FTS 355-2749

LOUISIANA
P.O. Box 1630
3737 Government Street
Alexandria, LA 71301

(318) 473-7751
FTS 497-7751

MAINE
USDA Building
University of Maine
Orono, ME 04473

(207) 866-2132
FTS 833-7393

MARYLAND
Hartwick Bldg.
Room 522
4321 Hartwick Road
College Park, MD 20740

(301) 344-4180
FTS 344-4180

MASSACHUSETTS
451 West Street
Amherst, MA 01002

(413) 256-0441
FTS 256-0441

MICHIGAN
Room 101
1405 S. Harrison Road
East Lansing, MI 48823

(517) 337-6702
FTS 374-6702

MINNESOTA
200 Federal Building &
U.S. Courthouse
316 N. Robert Street
St. Paul, MN 55101

(612) 725-7675
FTS 725-7675

MISSISSIPPI
100 W. Capitol
Suite 1321
Federal Building
Jackson, MS 39269

(601) 969-5205
FTS 490-5205

MISSOURI
555 Vandiver Drive
Columbia, MO 65202

(314) 875-5214
FTS 276-5214

MONTANA
32 E. Babcock
P.O. Box 970
Bozeman, MT 59715

(406) 587-5271
FTS 585-4322

NEBRASKA
Federal Building, Rm 345
U.S. Courthouse
100 Centennial Mall, North
P.O. Box 82502
Lincoln, NE 68501

(402) 471-5300
FTS 541-5300

NEVADA
U.S. Post Office Bldg.
50 S. Virginia St.
P.O. Box 4850
Reno, NV 89505

(702) 784-5863
FTS 470-5863

NEW HAMPSHIRE
Federal Building
P.O. Box G
Durham, NH 03824

(603) 868-7581
FTS 834-0505

NEW JERSEY
1370 Hamilton Street
Somerset, NJ 08873

(201) 246-1205
FTS 342-5341

NEW MEXICO
P.O. Box 2007
517 Gold Avenue, SW., Rm 301
Albuquerque, NM 87103

(404) 766-2173
FTS 474-2173

NEW YORK
U.S. Courthouse & Federal Bldg.
100 S. Clinton Street
Room 771
Syracuse, NY 13260

(315) 423-5521
FTS 950-5521

NORTH CAROLINA
310 New Bern Avenue
Room 544
Federal Office Building
P.O. Box 27301
Raleigh, NC 27611

(919) 755-4210
FTS 672-4210

NORTH DAKOTA
Federal Building, Rm 270
Rosser Ave. & Third St.
P.O. Box 1458
Bismarck, ND 58502

(701) 255-4011, x-421
FTS 783-4421

OHIO
Room 522
Federal Building
200 North High Street
Columbus, OH 43215

(614) 469-6962
FTS 943-6962

OKLAHOMA
Agricultural Ctr. Bldg.
Farm Road & Brumley Street
Stillwater, OK 74074

(405) 626-4360
FTS 728-4360

OREGON
1220 SW Third Avenue
Federal Building, 16th Floor
Portland, OR 97204

(503) 221-2751
FTS 423-2751

PENNSYLVANIA
Federal Building &
U.S. Courthouse
Box 985
Federal Square Station
Harrisburg, PA 17108

(717) 782-2202
FTS 590-2202

PUERTO RICO
Federal Building, Rm 639
Chardon Avenue
GPO Box 4868
San Juan, PR 00936

(809) 753-4206
FTS 753-4206

RHODE ISLAND
46 Quaker Lane
West Warwick, RI 02893

(401) 828-1300
FTS 828-4654

SOUTH CAROLINA
Federal Bldg., Rm 950
1835 Assembly St.
Columbia, SC 29210

(803) 765-5681
FTS 677-5681

SOUTH DAKOTA
Federal Building, Rm 203
200 4th Street, SW
Huron, SD 57350

(605) 352-8651
FTS 782-2333

TENNESSEE
U.S. Courthouse, Rm 675
801 Broadway Street
Nashville, TN 37203

(615) 251-5471
FTS 852-5471

TEXAS
Federal Bldg.
101 S. Main Street
P.O. Box 648
Temple, TX 76503

(817) 774-1214
FTS 736-1214

UTAH
4012 Federal Building
125 S. State Street
P.O. Box 11350
Salt Lake City, UT 84147

(801) 524-5050
FTS 588-5050

VERMONT
1 Burlington Square
Suite 205
Burlington, VT 05401

(802) 951-6795
FTS 832-6795

VIRGINIA
P.O. Box 10026
Federal Building, Rm 9201
400 N 8th Street
Richmond, VA 23240

(804) 771-2457
FTS 925-2457

Soil Conservation Service, U.S. Department of Agriculture (continued)

WASHINGTON
360 U.S. Courthouse
W. 920 Riverside Avenue
Spokane, WA 99201

(509) 456-3711
FTS 439-3711

WISCONSIN
4601 Hammersley Road
Madison, WI 53711

(608) 264-5351
FTS 364-5351

WEST VIRGINIA
75 High Street, Rm 301
Morgantown, WV 26505

(304) 291-4151
FTS 923-4151

WYOMING
Federal Office Building
100 East "B" Street
Casper, WY 82601

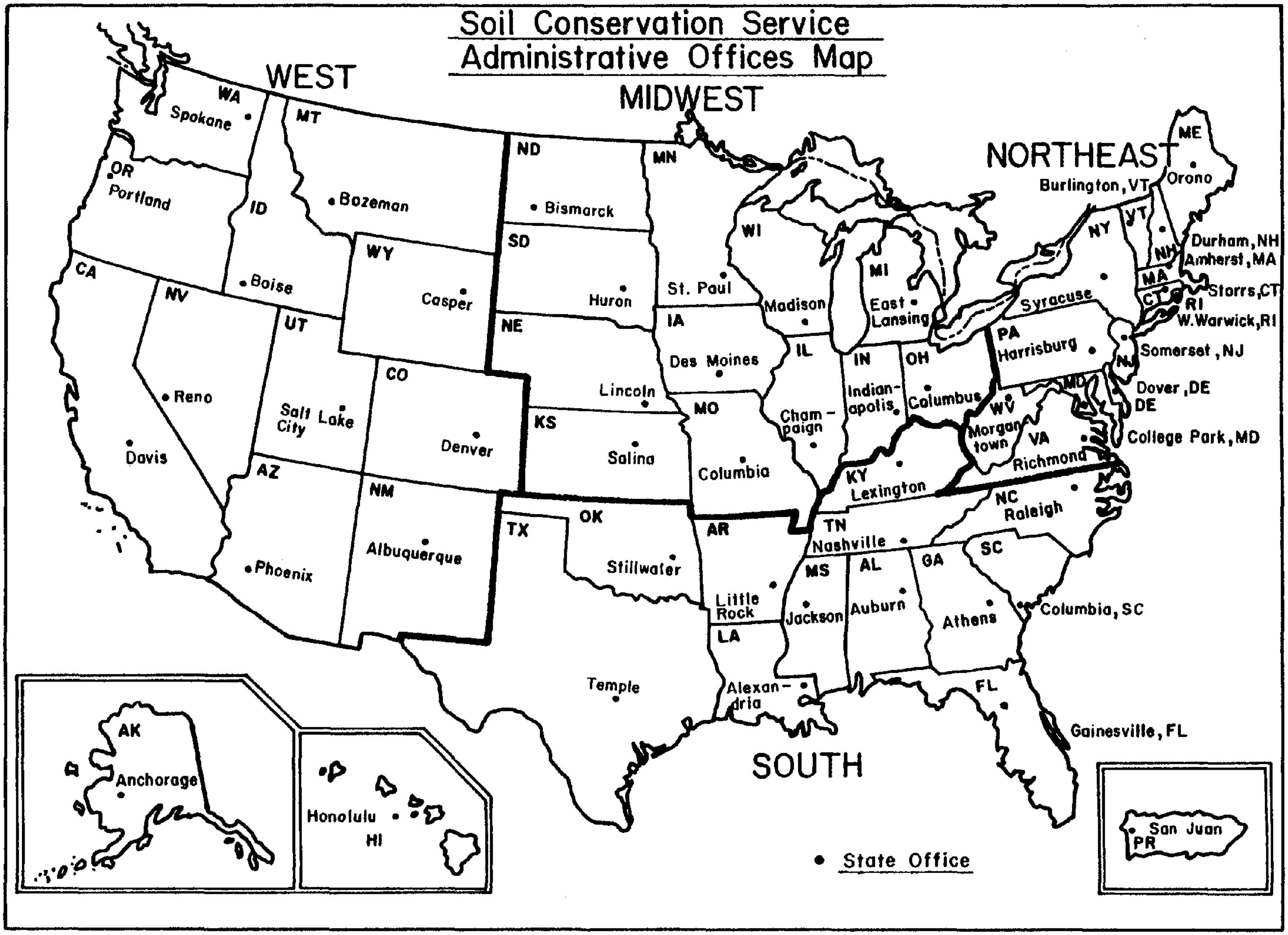
(307) 261-5201
FTS 328-5201

---

Economic Research Service, U.S. Department of Agriculture

Natural Resource Economic Division
Water Branch
500 12th St. SW - Rm 428
Washington, DC 20250

(202) 447-8320


Soil Conservation Service
Administrative Offices Map
WEST
MIDWEST
NORTHEAST
SOUTH
WA
Spokane
OR
Portland
CA
Davis
NV
Reno
ID
Boise
MT
Bozeman
WY
Casper
UT
Salt Lake City
AZ
Phoenix
CO
Denver
NM
Albuquerque
ND
Bismarck
SD
Huron
NE
Lincoln
KS
Salina
OK
Stillwater
TX
Temple
MN
St. Paul
IA
Des Moines
MO
Columbia
AR
Little Rock
LA
Alexandria
WI
Madison
IL
Champaign
MI
East Lansing
IN
Indianapolis
OH
Columbus
KY
Lexington
TN
Nashville
MS
Jackson
AL
Auburn
GA
Athens
FL
Gainesville, FL
SC
Columbia, SC
NC
Raleigh
VA
Richmond
WV
Morgantown
PA
Harrisburg
NY
Syracuse
ME
Orono
VT
Burlington, VT
NH
Durham, NH
MA
Amherst, MA
CT
Storrs, CT
RI
W. Warwick, RI
NJ
Somerset, NJ
DE
Dover, DE
MD
College Park, MD
AK
Anchorage
HI
Honolulu
PR
San Juan
State Office

## Corps of Engineers, Department of the Army

### New England Division

There are no district offices in this Division.

U.S. Army Engineer Division,
New England
424 Trapelo Road
Waltham, MA 02254

(617) 647-8220
FTS 839-7220

---

### North Atlantic Division

U.S. Army Engineer Division,
North Atlantic
90 Church Street
New York, NY 10007

(212) 264-7101
FTS 8-264-7101

U.S. Army Engineer District,
Baltimore
31 Hopkins Plaza
P.O. Box 1715
Baltimore, MD 21203

(301) 962-4545
FTS 922-4545

U.S. Army Engineer District,
New York
26 Federal Plaza
New York, NY 10278

(212) 264-0100
FTS 264-0100

U.S. Army Engineer District,
Norfolk
803 Front Street
Norfolk, VA 23510

(804) 441-3601
FTS 827-3601

U.S. Army Engineer District,
Philadelphia
2nd and Chestnut Streets
Philadelphia, PA 19106

(215) 597-4848
FTS 597-4848

## South Atlantic Division

U.S. Army Engineer Division,
South Atlantic
510 Title Building
30 Prior Street, SW
Atlanta, GA 30303

(404) 221-6711
FTS 242-6711

U.S. Army Engineer District,
Charleston
Federal Building
334 Meeting Street
P.O. Box 919
Charleston, SC 29402

(803) 724-4229
FTS 677-4229

U.S. Army Engineer District,
Savannah
200 E. Saint Julian Street
P.O. Box 899
Savannah, GA 31402

(912) 944-5224
FTS 248-5224

U.S. Army Engineer District,
Jacksonville
400 West Bay Street
P.O. Box 4970
Jacksonville, FL 32232

(904) 791-2241
FTS 946-2241

U.S.Army Engineer District,
Wilmington
308 Federal Building
P.O. Box 1890
Wilmington, NC 28402

(919) 343-4501
FTS 671-4647

U.S. Army Engineer District,
Mobile
109 St. Joseph Street
P.O. Box 2288
Mobile, AL 36628

(205) 690-2511
FTS 537-2511

## Ohio River Division

U.S. Army Engineer Division,
Ohio River
550 Main Street
P.O Box 1159
Cincinnati, OH 45201

(513) 684-3002
FTS 684-3002

U.S. Army Engineer District,
Huntington
502 Eighth Street
P.O. Box 2127
Huntington, WV 25721

(304) 529-5395
FTS 924-5395

U.S. Army Engineer District,
Louisville
600 Federal Place
P.O. Box 59
Louisville, KY 40201

(502) 582-5601
FTS 352-5601

U.S. Army Engineer District,
Nashville
801 Broadway
P.O. Box 1070
Nashville, TN 37202

(615) 251-5626
FTS 852-5626

U.S. Army Engineer District,
Pittsburgh
Federal Building
1000 Liberty Avenue
Pittsburgh, PA 15222

(412) 644-6800
FTS 722-6800

## North Central Division

U.S. Army Engineer Division,
North Central
536 South Clark Street
Chicago, IL 60605

(312) 353-6310
FTS 353-6310

## North Central Division (continued)

U.S. Army Engineer District,
Buffalo
1776 Niagara Street
Buffalo, NY 14207

(716) 876-5454, x-2000
FTS 473-2200

U.S. Army Engineer District,
Chicago
219 S. Dearborn Street
Chicago, IL 60604

(312) 353-6400
FTS 353-6400

U.S. Army Engineer District,
Detroit
477 Michigan Ave
P.O. Box 1027
Detroit, MI 48231

(313) 226-6762
FTS 226-6762

U.S. Army Engineer District
Rock Island
Clock Tower Building
Rock Island, IL 61201

(309) 788-6361, x-6224
FTS 386-6011

U.S. Army Engineer District
St. Paul
1135 U.S. Post Office and
Custom House
St. Paul, MN 55101

(612) 725-7501
FTS 725-7501

---

## Lower Mississippi Valley Division

U.S. Army Engineer Division,
Lower Mississippi Valley
1400 Walnut Street
P.O. Box 80
Vicksburg, MS 39180

(601) 634-5750
FTS 542-5750

U.S. Army Engineer District,
Memphis
B-314 Clifford Davis
Federal Building
Memphis, TN 38103

(901) 521-3221
FTS 222-3221

U.S. Army Engineer District,
St. Louis
210 Tucker Blvd. N.
St. Louis, MO 63101

(314) 263-5660
FTS 273-5660

## Lower Mississippi Valley Division (continued)

U.S. Army Engineer District,
New Orleans
Foot of Prytania Street
P.O. Box 60267
New Orleans, LA 70160

(504) 838-2204
FTS 687-2204

U.S. Army Engineer District,
Vicksburg
U.S. Post Office & Courthouse
P.O. Box 60
Vicksburg, MS 39180

(601) 634-5010
FTS 542-5010

---

## Missouri River Division

U.S. Army Engineer Division,
Missouri River
12565 West Center Road
P.O. Box 103 Downtown Station
Omaha, NE 68101

(402) 221-7201
FTS 864-7201

U.S. Army Engineer District,
Kansas City
700 Federal Building
601 E. 12th Street
Kansas City, MO 64106

(816) 374-3201
FTS 758-3201

U.S. Army Engineer District,
Omaha
215 North 17th Street
Rm. 6014 U.S. Post Office and
Courthouse
Omaha, NE 68102

(402) 221-3900
FTS 864-3900

---

## Southwestern Division

U.S. Army Engineer Division,
Southwestern
1114 Commerce St.
Dallas, TX 75242

(214) 767-2500
FTS 729-2500

## Southwestern Division (continued)

U.S. Army Engineer District,
Albuquerque
517 Gold Avenue, S.W.
Albuquerque, NM 87103

(505) 766-2732
FTS 474-2732

U.S. Army Engineer District,
Fort Worth
819 Taylor Street
P.O. Box 17300
Fort Worth, TX 76102

(817) 334-2300
FTS 334-2300

U.S. Army Engineer District,
Galveston
110 Essayons Bldg.
400 Barracuda Avenue
P.O. Box 1229
Galveston, TX 77553

(713) 766-3006
FTS 527-6006

U.S. Army Engineer District,
Little Rock
700 W. Capitol
P.O. Box 867
Little Rock, AR 72203

(501) 378-5531
FTS 740-5531

U.S. Army Engineer District,
Tulsa
224 South Boulder
P.O. Box 61
Tulsa, OK 74121

(918) 581-7311
FTS 736-7311

---

## North Pacific Division

U.S. Army Engineer Division,
North Pacific
220 N.W. 8th Avenue
P.O. Box 2870
Portland, OR 97208

(503) 221-3700
FTS 423-3700

## North Pacific Division (continued)

U.S. Army Engineer District,
Alaska
Building 21-700
Pouch 898
Elmendorf AFB
Anchorage, AK 99506

(907) 279-1132
FTS 8-907-279-1132

U.S. Army Engineer District,
Seattle
4735 E. Marginal Way South
P.O. Box C-3755
Seattle, WA 98124

(206) 764-3690
FTS 399-3690

U.S. Army Engineer District,
Portland
319 S.W. Pine
P.O. Box 2946
Portland, OR 97208

(503) 221-6000
FTS 423-6000

U.S. Army Engineer District,
Walla Walla
Building 602
City-County Airport
Walla Walla, WA 99362

(509) 525-5500, x-100
FTS 442-5100

---

## South Pacific Division

U.S. Army Engineer Division,
South Pacific
600 Sansome Street, Rm. 1216
San Francisco, CA 94111

(415) 446-0914
FTS 446-0914

U.S. Army Engineer District,
Los Angeles
300 N. Los Angeles Street
P.O. Box 2711
(415) 974-0358
Los Angeles, CA 90053

(213) 688-5300
FTS 798-5300

U.S. Army Engineer District,
San Francisco
211 Main Street
San Francisco, CA 94105

FTS 974-0429

U.S. Army Engineer District,
Sacramento
650 Capitol Mall
Sacramento, CA 95814

(916) 440-2232
FTS 448-2232


Divisions for Civil Works Activities
U.S. Army Corps of Engineers
NORTH CENTRAL
NORTH PACIFIC
MISSOURI RIVER
SOUTH PACIFIC
SOUTHWESTERN
LOWER MISSISSIPPI VALLEY
OHIO RIVER
SOUTH ATLANTIC
NORTH ATLANTIC
NEW ENGLAND
WA
OR
ID
MT
WY
ND
SD
NE
KS
CO
UT
NV
CA
AZ
NM
TX
OK
AR
LA
MO
IA
MN
WI
IL
MI
IN
OH
KY
TN
MS
AL
GA
FL
SC
NC
VA
WV
PA
NY
VT
NH
ME
MA
CT
RI
NJ
MD
DE
AK
HI
PR
Seattle
Walla Walla
Portland
Sacramento
San Francisco
Los Angeles
Albuquerque
Omaha
Kansas City
Tulsa
Dallas
Fort Worth
Galveston
Little Rock
St. Paul
Rock Island
Chicago
St. Louis
Detroit
Buffalo
Pittsburgh
Cincinnati
Louisville
Huntington
Nashville
Memphis
Vicksburg
New Orleans
Mobile
Atlanta
Savannah
Charleston
Wilmington
Jacksonville
Norfolk
Baltimore
Philadelphia
New York
Boston
Division Headquarters
District Headquarters
Division and District Headquarters

## National Oceanic and Atmospheric Administration, U.S. Department of Commerce

### National Weather Service

Eastern Region
585 Stewart Avenue
Garden City, NY 11530

(516) 228-5462
FTS 649-5462

Alaska Region
Box 23, 701 C Street
Anchorage, AK 99513

(907) 271-3477
FTS 271-3477

Southern Region
819 Taylor Street
10A29 Federal Office Building
Fort Worth, TX 76102

(817) 334-2674
FTS 334-2674

Pacific Region
300 Ala Moana Blvd
4110 Federal Building
P.O. Box 50027
Honolulu, HI 96850

(808) 546-5690
FTS 546-5690

Central Region
601 East 12th Street, Rm 1835
Kansas City, MO 64106

(816) 374-3229
FTS 758-3229

Western Region
Box 11188 Federal Building
125 South State Street
Salt Lake City, UT 84147

(801) 524-5137
FTS 588-5137

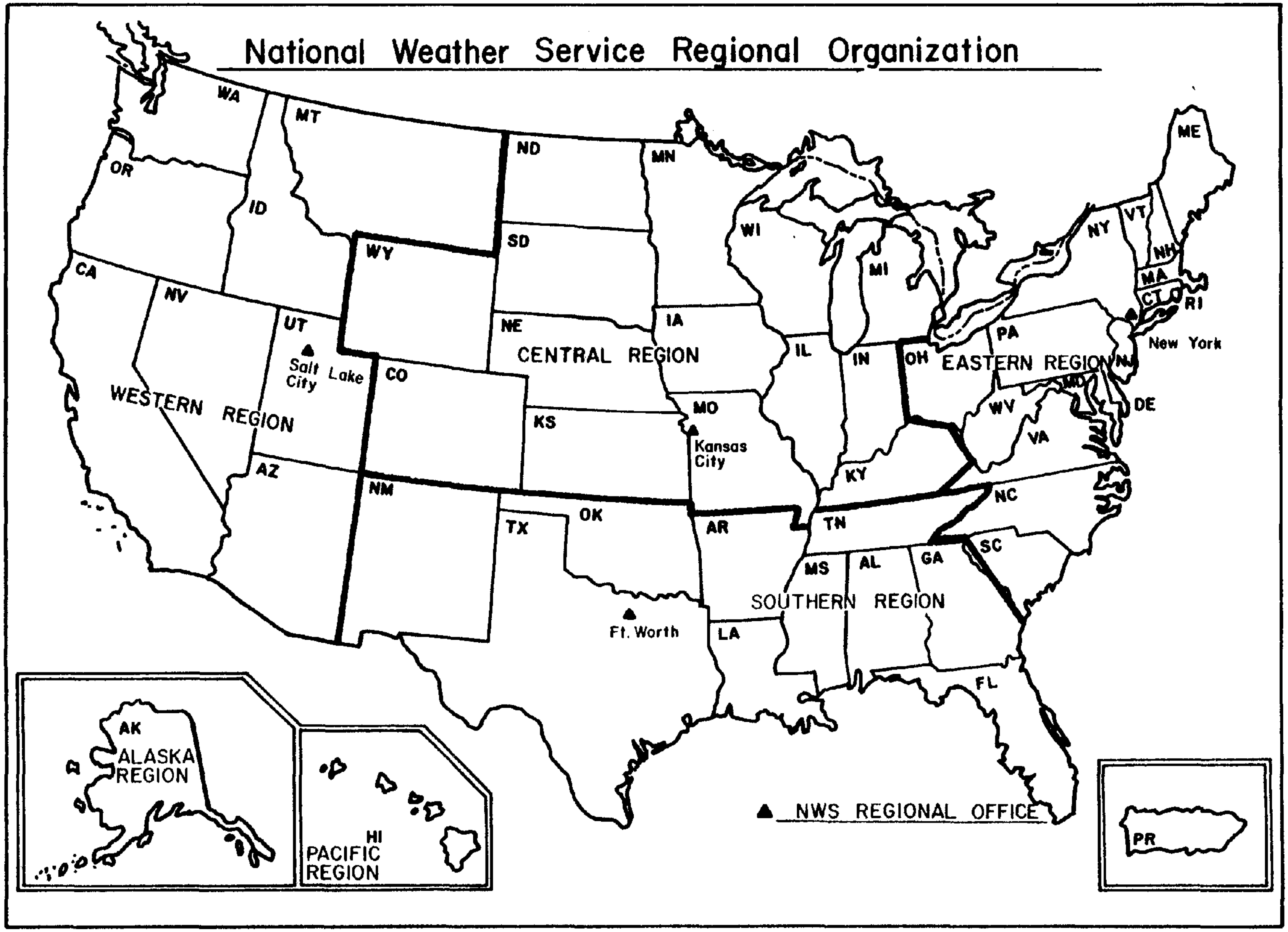
### Environmental Data and Information Service
(National Environmental Satellite, Data, and Information Service)

National Climatic Data Center
Federal Building
Asheville, NC 28801

(704) 259-0682
FTS 672-0682

National Oceanographic Data Center
2001 Wisconsin Avenue, NW., Page Bldg. 1
Washington, DC 20235

(202) 634-7510
FTS 634-7510


National Weather Service Regional Organization
WA
MT
ND
MN
OR
ID
WY
SD
WI
MI
CA
NV
UT
NE
IA
Salt Lake City
CO
CENTRAL REGION
IL
IN
OH
EASTERN REGION
WESTERN REGION
KS
MO
Kansas City
KY
WV
VA
AZ
NM
OK
TX
AR
TN
NC
SC
MS
AL
GA
SOUTHERN REGION
Ft. Worth
LA
FL
ME
VT
NY
NH
MA
CT
RI
PA
NJ
MD
DE
New York
AK
ALASKA REGION
HI
PACIFIC REGION
NWS REGIONAL OFFICE
PR

## Federal Power Marketing Administrations, U.S. Department of Energy

Southeastern Power Administration
Samuel Elbert Bldg.
Elberton, GA 30635

(404) 283-3261

Bonneville Power Administration
P.O. Box 3621
Portland, OR 97208

(503) 234-3361
FTS 429-3361

Western Area Power Administration
P.O. Box 3402
Golden, CO 80401

(303) 231-1511
FTS 327-1511

Alaska Power Administration
P.O. Box 50
Juneau, AK 99802

(907) 586-7405

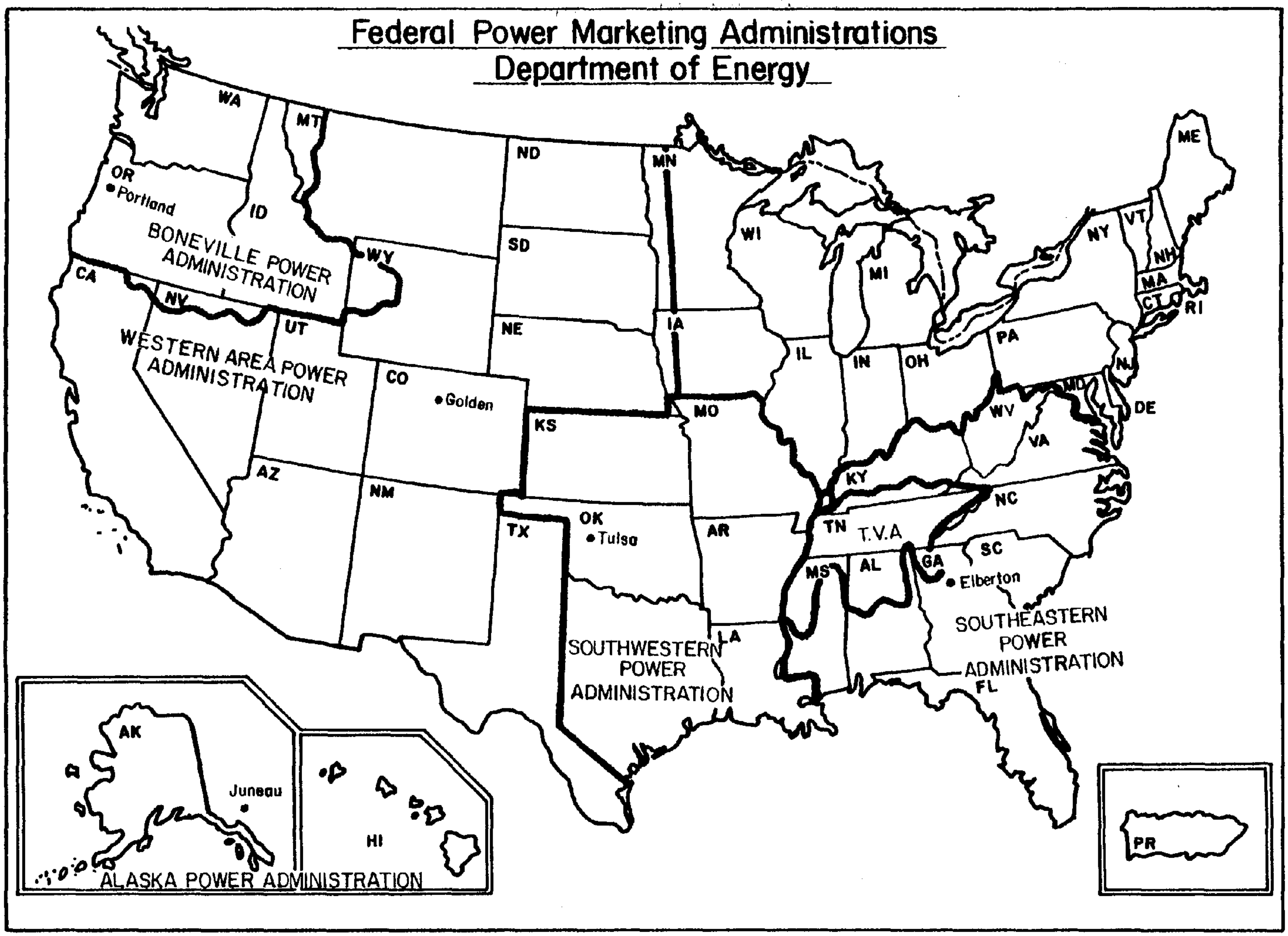
Southwestern Power Administration
P.O. Drawer 1619
Tulsa, OK

(918) 581-7474
FTS 745-7474

## Tennessee Valley Authority

400 Commerce Avenue
Knoxville, TN 37902

(615) 632-3871




Federal Power Marketing Administrations
Department of Energy

U.S. Environmental Protection Agency

REGION I
Kennedy Fed. Bldg., Rm 2203
Boston, MA 02203

(617) 223-7210
FTS 8-223-7210

REGION II
26 Federal Plaza, Rm 900
New York, NY 10278

(212) 264-2525
FTS 8-264-2525

REGION III
6th and Walnut Streets
Philadelphia, PA 19106

(215) 597-9800
FTS 8-597-9800

REGION IV
345 Courtland Street, NE
Atlanta, GA 30365

(404) 881-4727
FTS 8-257-4727

REGION V
230 S. Dearborn Street
Chicago, IL 60604

(312) 353-2000
FTS 8-353-2000

REGION VI
1201 Elm Street
Dallas, TX 75270

(214) 767-2600
FTS 8-729-2600

REGION VII
324 E. 11th Street
Kansas City, MO 64106

(214) 374-5493
FTS 8-758-5493

REGION VIII
1860 Lincoln Street
Denver, CO 80295

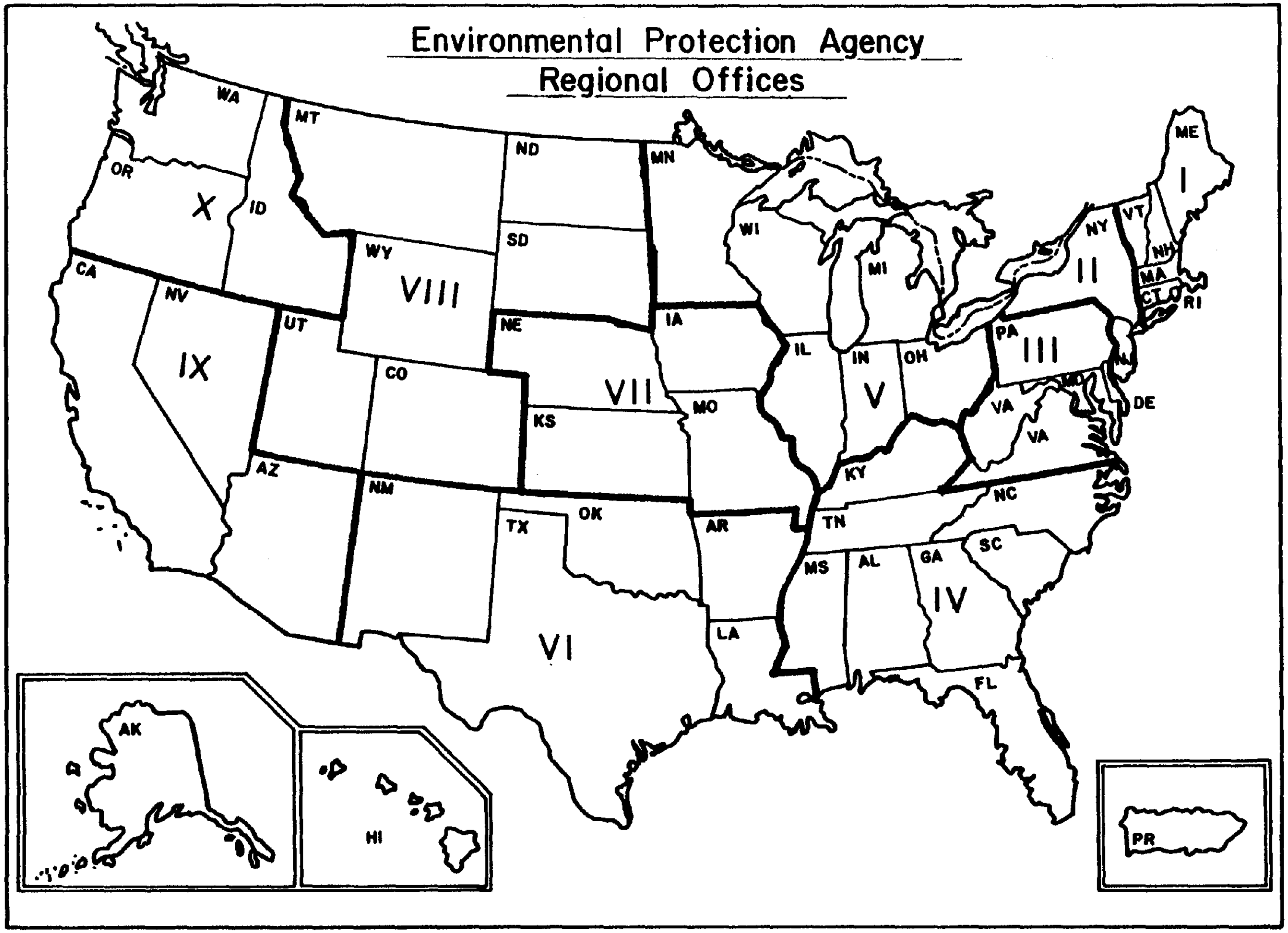
(303) 837-3895
FTS 8-327-3895

REGION IX
215 Freemont Street
San Francisco, CA 94105

(415) 974-8153
FTS 8-454-8153

REGION X
1200 6th Avenue
Seattle, WA 98101

(206) 442-5810
FTS 8-399-5810


Environmental Protection Agency
Regional Offices
WA
OR
X
ID
MT
ND
SD
WY
VIII
MN
WI
MI
NY
VT
ME
I
NH
MA
CT
RI
II
CA
NV
IX
UT
CO
NE
VII
IA
IL
IN
V
OH
PA
III
NJ
MD
DE
VA
VA
KS
MO
KY
AZ
NM
TX
OK
AR
TN
NC
SC
MS
AL
GA
IV
VI
LA
FL
AK
HI
PR

Federal Emergency Management Agency

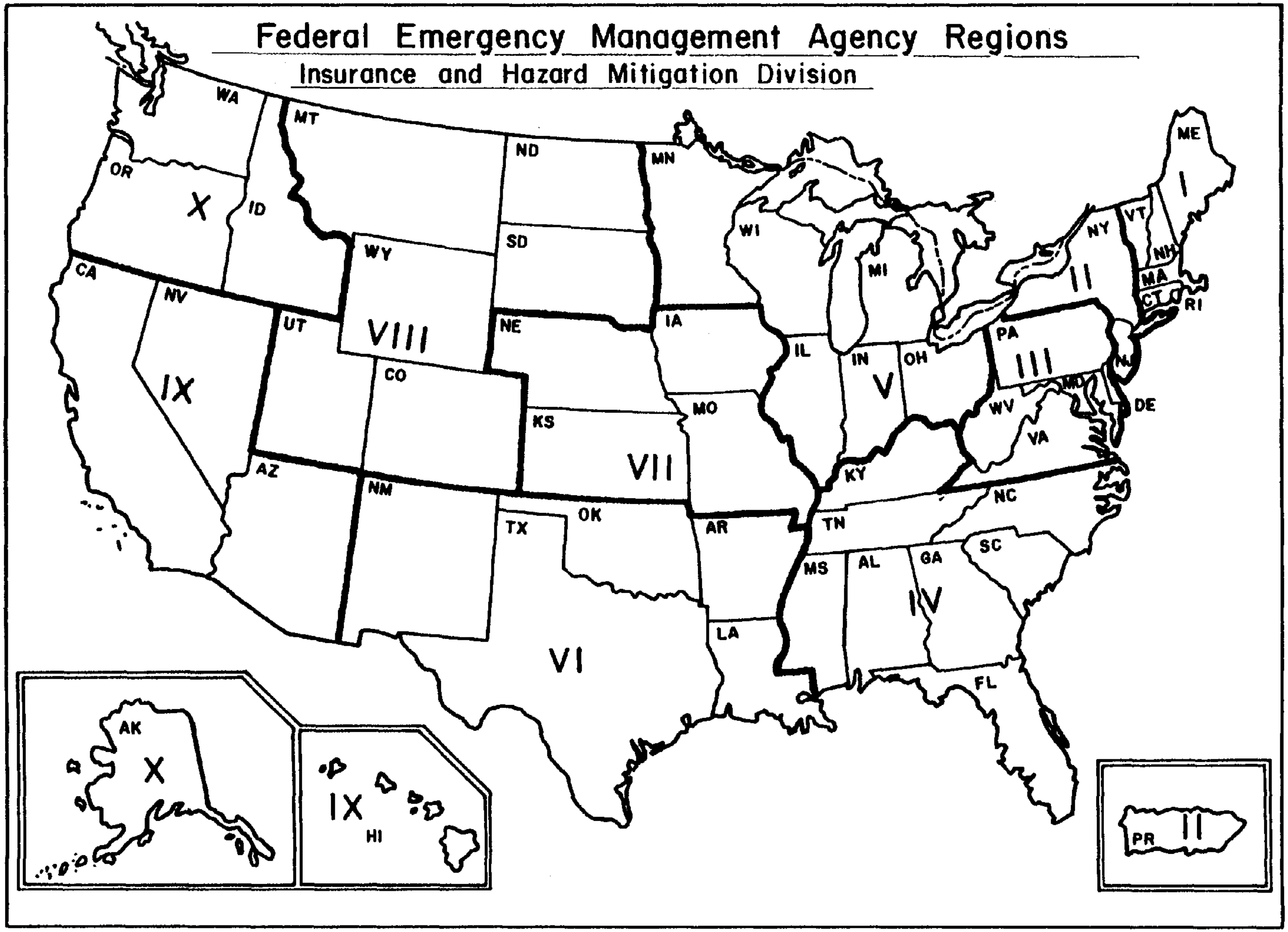
Insurance and Hazard Mitigations Division

| | |
|---|---|
| REGION I | J. W. McCormack, POCH<br>Boston, MA 02109<br><br>(617) 223-4741<br>FTS 8-223-4741 |
| REGION II | 26 Federal Plaza - Rm 1349<br>New York, NY 10278<br><br>(212) 264-8980<br>FTS 8-264-8980 |
| REGION III | Curtis Building, 17th Floor<br>6th and Walnut Street<br>Philadelphia, PA 19106<br><br>(215) 597-9416<br>FTS 8-597-9416 |
| REGION IV | Gulf Oil Bldg.<br>1375 Peachtree Street, NE<br>Atlanta, GA 30309<br><br>(404) 881-2400<br>FTS 8-257-2400 |
| REGION V | 300 South Wacker Drive, 24th Floor<br>Chicago, IL 60606<br><br>(312) 353-1500<br>FTS 8-353-8661 |
| REGION VI | Federal Regional Center<br>800 North Loop 288<br>Denton, TX 76201<br><br>(817) 387-5811<br>FTS 8-749-9201 |

Federal Emergency Management Agency (continued)

Insurance and Hazard Mitigations Division (contined)

| | |
|---|---|
| REGION VII | Federal Office Building<br>911 Walnut Street, Rm 300<br>Kansas City, MO 64106<br><br>(816) 374-5912<br>FTS 8-758-5912 |
| REGION VIII | Building 710<br>Federal Regional Center<br>Denver, CO 80225<br><br>(303) 234-6542<br>FTS 8-234-2553 |
| REGION IX | Bldg. 305<br>Presidio of San Franciso, CA 94129<br><br>(415) 556-8794<br>FTS 8-556-8794 |
| REGION X | Federal Regional Center<br>Bothell, WA 98021<br><br>(206) 481-8800<br>FTS 8-396-0284 |


Federal Emergency Management Agency Regions
Insurance and Hazard Mitigation Division
WA
OR
ID
X
MT
WY
ND
SD
NE
MN
IA
WI
MI
IL
IN
OH
NY
VT
NH
ME
MA
CT
RI
PA
NJ
MD
DE
WV
VA
KY
TN
NC
SC
GA
AL
MS
FL
AR
LA
MO
KS
OK
TX
NM
CO
UT
AZ
NV
CA
I
II
III
IV
V
VI
VII
VIII
IX
AK
X
IX
HI
PR
II

## Bureau of Land Management, U.S. Department of the Interior

### ALASKA

701 'C' Street
Box 13
Anchorage, AL 99513

(907) 271-5076

### ARIZONA

3707 North 7th Street
Phoenix, AZ 85014

FTS 261-3873

### CALIFORNIA

Federal Office Bldg.
Room E-2841
2800 Cottage Way
Sacramento, CA 95825

(916) 484-4676
FTS 468-4676

### COLORADO

2020 Arapahoe Street
Denver, CO 80205

(303) 837-4325
FTS 327-4325

### EASTERN STATES OFFICE

350 So. Pickett Street
Alexandria, VA 22304

(703) 235-2833
FTS 235-2833

### IDAHO

3380 Americana Terrace
Boise, ID 83706

(208) 334-1401
FTS 554-1401

### MONTANA

Granite Tower
222 N. 32nd Street
P.O. Box 36800
Billings, MT 59107

(406) 657-6461
FTS 585-6461

### NEVADA

Federal Building
Room 3008
300 Booth St.
P.O. Box 12000
Reno, NV 89520

(702) 784-5451
FTS 470-5451

### NEW MEXICO

Joseph M. Montoya
Federal Building
South Federal Place
P.O. Box 1449
Santa Fe, NM 87501

(505) 988-6030
FTS 476-6030

OREGON

825 N.E. Multnomah St.
P.O. Box 2965
Portland, OR 97208

(503) 231-6251
FTS 429-6251

UTAH

University Club Bldg.
136 East South Temple
Salt Lake City, UT 84111

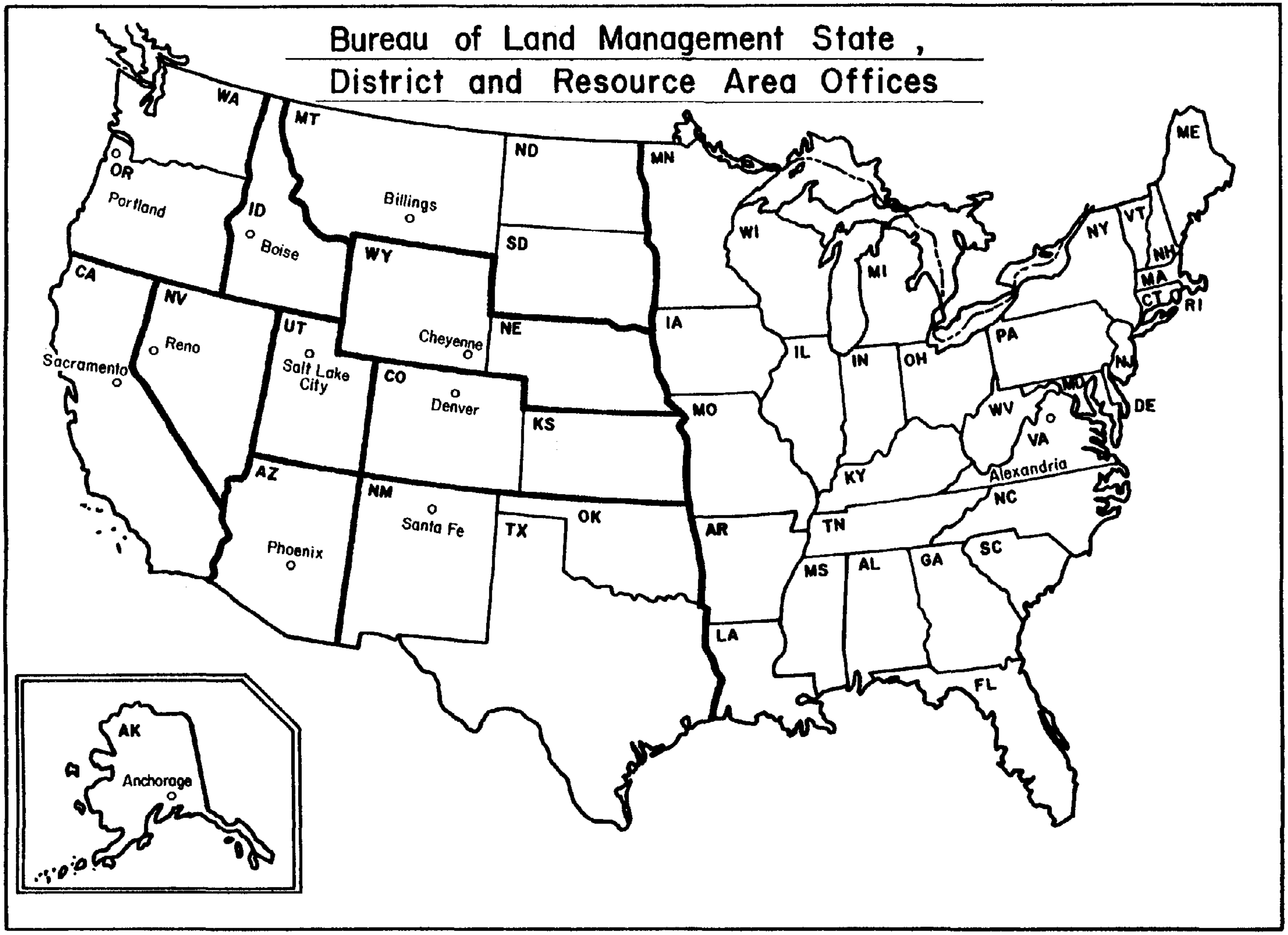
(801) 524-5311
FTS 588-5311

WYOMING

2515 Warren Avenue
P.O. Box 1828
Cheyenne, WY 82001

(307) 772-2326
FTS 328-2326

# Bureau of Land Management State, District and Resource Area Offices



## Bureau of Reclamation, U.S. Department of the Interior

PACIFIC NORTHWEST REGION
Federal Building, U.S. Courthouse
550 West Fort Street
Boise, ID 83724

(208) 384-1908

MID-PACIFIC REGION
Federal Office Building
2800 Cottage Way
Sacramento, CA 95825

(916) 484-4571

LOWER COLORADO REGION
Nevada Hwy. & Park Street
P.O. Box 427
Boulder City, NV 89005

(702) 293-8000

UPPER COLORADO REGION
125 South State Street
P.O. Box 11568
Salt Lake City, UT 84147

(801) 524-5566

SOUTHWEST REGION
Commerce Building, Suite 201
714 South Tyler Street
Amarillo, TX 79101

(806) 378-5400

UPPER MISSOURI REGION
Federal Office Building
316 North 26th Street
P.O. Box 2553
Billings, MT 59103

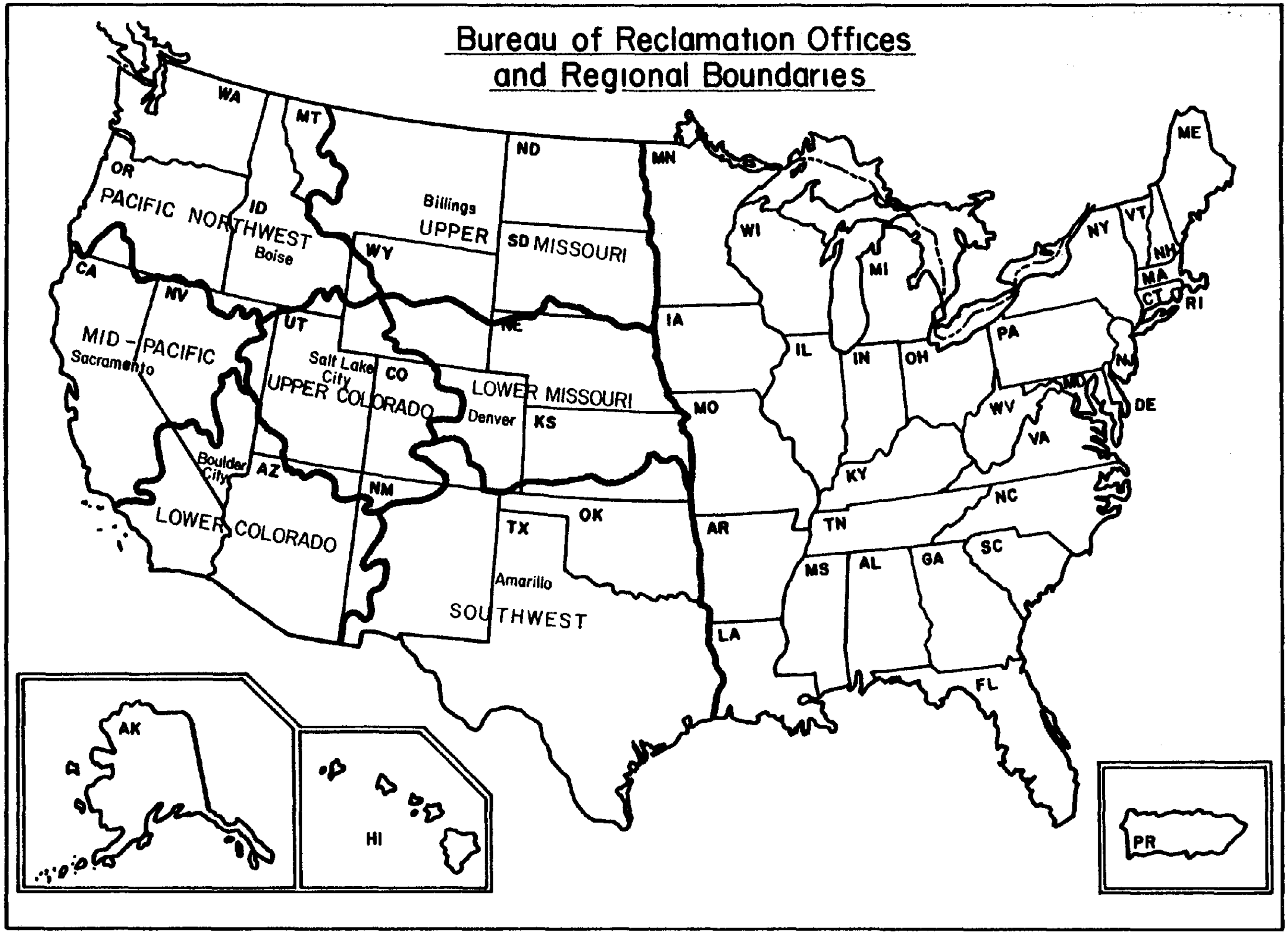
(406) 657-6214

LOWER MISSOURI REGION
Building 20
P.O. Box 25247
Denver Federal Center
Denver, CO 80225

(303) 234-4441

ENGINEERING AND RESEARCH CENTER
P.O. Box 25007
Denver Federal Center
Denver, CO 80225

(303) 234-2041




Bureau of Reclamation Offices and Regional Boundaries

Geological Survey, U.S. Department of the Interior

A. General Hydrologic Information

Hydrologic Information Unit
U.S. Geological Survey
419 National Center
Reston, VA 22092

(703) 860-7521
FTS 928-7521

B. Hydrologic Information for a specific area, contact the USGS District Office listed below:

ALABAMA
520 19th Avenue
Tuscaloosa, AL 35401

(205) 752-8104
FTS 229-2957

ALASKA
1515 East 13th Avenue
Anchorage, AK 99501

(907) 271-4138
FTS (907) 271-4138

ARIZONA
Federal Building, FB 44
301 West Congress Street
Tucson, AZ 85701

(602) 629-6671
FTS 762-6671

ARKANSAS
Rm. 2301 Federal Office Bldg.
700 West Capitol Avenue
Little Rock, AR 72201

(501) 378-6391
FTS 740-6391

CALIFORNIA
Rm. W-2235 Federal Building
2800 Cottage Way
Sacramento, CA 95825

(916) 484-4606
FTS 468-4606

COLORADO
Box 25046, Mail Stop 415
Denver Federal Center
Lakewood, CO 80225

(303) 234-5092
FTS 234-5092

CONNECTICUT
See listing for Massachusetts

DELAWARE
See listing for Maryland

DISTRICT OF COLUMBIA
See listing for Maryland

FLORIDA
Hobbs Federal Building
Suite 3015
Tallahassee, FL 32301

(904) 681-7620
FTS 965-7620

GEORGIA
6481 Peachtree Industrial Blvd.
Suite B
Doraville, GA 30360

(404) 221-4858
FTS 242-4858

HAWAII
P.O. Box 50166
300 Ala Moana Blvd.
Room 6110
Honolulu, HI 96850

(208) 546-8331
FTS (808) 546-8331

IDAHO
230 Collins Road
Boise, ID 83702

(208) 334-1750
FTS 554-1750

ILLINOIS
Champaign County Bank Plaza
102 East Main, 4th Floor
Urbana, IL 61801

(217) 398-5353
FTS 958-5353

INDIANA
6023 Guion Road
Suite 201
Indianapolis, IN 46254

(317) 927-8640
FTS 336-8640

IOWA
P.O. Box 1230
Rm. 269 Federal Building
400 South Clinton Street
Iowa City, IA 52244

(319) 337-4191
FTS 863-6521

KANSAS
1950 Constant Avenue-Campus West
University of Kansas
Lawrence, KS 66044

(913) 864-4321
FTS 752-2301

KENTUCKY
Rm. 572 Federal Building
600 Federal Place
Louisville, KY 40202

(502) 582-5241
FTS 352-5241

LOUISIANA
P.O. Box 66492
6554 Florida Boulevard
Baton Rouge, LA 70896

(504) 389-0281
FTS 687-0281

MAINE
See listing for Massachusetts

MARYLAND
208 Carrol Building
8600 LaSalle Road
Towson, MD 21204

(301) 828-1535
FTS 922-7872

MASSACHUSSETTS
150 Causeway Street
Suite 1309
Boston, MA 02114

(617) 223-2822
FTS 223-2822

MICHIGAN
6520 Mercantile Way
Suite 5
Lansing, MI 48910

(517) 377-1608
FTS 374-1608

MINNESOTA
Rm 702 Post Office Building
St. Paul, MN 55101

(612) 725-7841
FTS 725-7841

MISSISSIPPI
100 West Capitol Street
Suite 710, Federal Building
Jackson, MS 39269

(601) 960-4600
FTS 490-4600

MISSOURI
1400 Independence Road
Mail Stop 200
Rolla, MO 65401

(314) 341-0824
FTS 227-0824

MONTANA
301 South Park Avenue
Rm. 428 Federal Building
Drawer 10076
Helena, MT 59626

(406) 449-5302
FTS 585-5302

NEBRASKA
Rm. 406 Federal Building
& U.S. Courthouse
100 Centennial Mall North
Lincoln, NE 68508

(402) 471-5082
FTS 541-5082

NEVADA
See listing for Idaho

NEW HAMPSHIRE
See listing for Massachusetts

NEW JERSEY
Rm 430, Federal Building
402 East State Street
Trenton, NJ 08608

(609) 989-2162
FTS 483-2162

NEW MEXICO
Rm. 720, Western Bank Building
505 Marquette, Northwest
Albuquerque, NM 87102

(505) 766-2246
FTS 474-2246

NEW YORK
P.O. Box 1350
Rm. 343 U.S. Post Office
& Courthouse
Albany, NY 12201

(518) 472-3107
FTS 562-3107

NORTH CAROLINA
P.O. Box 2857
Rm. 436 Century Postal Station
Raleigh, NC 27602

(919) 755-4510
FTS 672-4510

NORTH DAKOTA
821 East Interstate Avenue
Bismark, ND 58501

(701) 255-4011, ex. 601
FTS 783-4601

OHIO
975 West Third Avenue
Columbus, OH 43212

(614) 469-5553
FTS 943-5553

OKLAHOMA
215 Dean A. McGee Avenue
Room 621
Oklahoma City, OK 73102

(405) 231-4256
FTS 736-4256

OREGON
847 NE 19th Avenue
Suite 300
Portland, OR 97232

(503) 231-2009
FTS 429-2009

PENNSYLVANIA
P.O. Box 1107
Federal Building, 4th Floor
228 Walnut Street
Harrisburg, PA 17108

(717) 782-4514
FTS 590-4514

PUERTO RICO
GSA Center, Building 652
GPO Box 4464
Highway 28, Pueblo Viejo
San Juan, PR 00936

(809) 783-4660
FTS (809) 753-4414

RHODE ISLAND
See listing for Massachusetts

SOUTH CAROLINA
1835 Assembly Street
Suite 658
Columbia, SC 29201

(803) 765-5966
FTS 677-5966

SOUTH DAKOTA
Rm. 317 Federal Building
200 Fourth Street, SW
Huron, SD 57350

(605) 352-8651, ex. 258
FTS 782-2258

TENNESSEE
Rm. A-413 Federal Building
& U.S. Courthouse
Nashville, TN 37203

(615) 251-5424
FTS 852-5424

TEXAS
Rm. 649 Federal Building
300 East Eighth Street
Austin, TX 78701

(512) 482-5766
FTS 770-5766

UTAH
Room 1016 Administration Bldg.
1745 West 1700 South
Salt Lake City, UT 84104

(801) 524-5663
FTS 588-5663

VERMONT
See listing for Massachusetts

VIRGINIA
See listing for Maryland

WASHINGTON
1201 Pacific Avenue
Suite 600
Tacoma, WA 98402

(206) 593-6510
FTS 390-6510

WEST VIRGINIA
603 Morris Street
Charleston, WV 25301

(304) 347-5130
FTS 930-5130

WISCONSIN
1815 University Avenue
Madison, WI 53705

(608) 262-2488
FTS 262-2488

WYOMING
Rm 4007 J.C. O'Mahoney Federal Center
2120 Capitol Avenue
Cheyenne, WY 82003

(307) 772-2153
FTS 328-2153

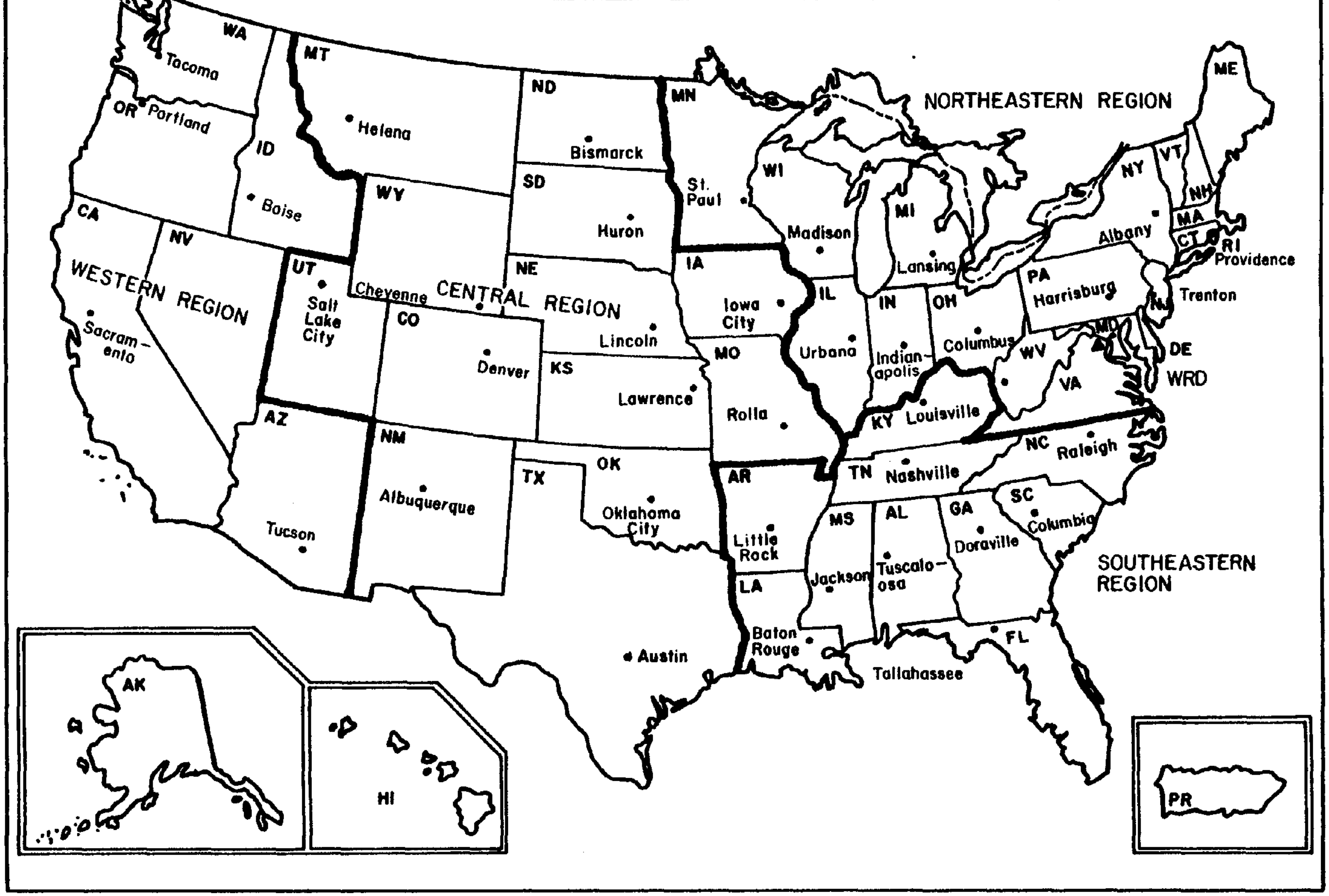
C. National Water Data Exchange (NAWDEX). NAWDEX is a confederation of Federal and non-Federal water oriented agencies working together to provide access to water data.

National Water Data Exchange
U.S. Geological Survey
421 National Center
Reston, VA 22092

(703) 860-6031
FTS 928-6031

D. Water Data Storage and Retrieval System (WATSTORE). Hydrologic data on ground water, surface water, and water quality are collected and stored on WATSTORE. Contact the appropriate USGS District Office listed above.

## U.S. Geological Survey District Offices



## Federal Highway Administration, U.S. Department of Transportation

### Direct Federal Divisions

Eastern Direct Federal Division
1000 North Glebe Road
Arlington, VA 22201

(703) 557-9070
FTS 557-9070

Central Direct Federal Division
555 Zang Street
P.O. Box 25406
Denver, CO 80225

(303) 234-4795
FTS 234-4795

Western Direct Federal Division
610 East Fifth Street
Vancouver, WA 98661

(206) 696-7710
FTS 422-7710

APPENDIX D

LIST OF REPORTS FOR ESTIMATING
RURAL DISCHARGES BY STATE


Alabama:

Hains, C.F., 1973, Floods in Alabama, magnitude and frequency: Alabama Highway Department, 174 p.

Olin, D.A., and Bingham, R.H., 1977, Flood frequency of small streams in Alabama: Alabama Highway Department HPR Report No. 83, Research Project 930-087.

Alaska:

Lamke, R.D., 1978, Flood characteristics of Alaskan streams: U.S. Geological Survey Water Resources Investigations 78-129.

Arizona:

Roeske, R.H., 1978, Methods for estimating the magnitude and frequency of floods in Arizona: Arizona Department of Transportation RS-15 (121), 82 p.

Arkansas:

Patterson, J. L., 1971, Floods in Arkansas, magnitude and frequency characteristics through 1968: Arkansas Geological Commissions, Water Resources Summary No. 11.

California:

Waananen, A.O., and Crippen, J.R., 1977, Magnitude and frequency of floods in California: U.S. Geological Survey Water-Resources Investigations 77-21 (PB-272 510/AS).

Colorado:

Hedman, E.R., Moore, D.O., and Livingston, R.K., 1972, Selected streamflow characteristics as related to channel geometry of perennial streams in Colorado: U.S. Geological Survey open-file report.

Livingston, R.K., 1980, Rainfall-runoff modeling and preliminary regional flood characteristics of small rural watersheds in the Arkansas River Basin in Colorado: U.S. Geological Survey Water-Resources Investigations 80-112.

McCain, J.R., and Jarrett, R.D., 1976, manual for estimating flood characteristics of natural-flow streams in Colorado: Colorado Water Conservation Board, Technical Manual no. 1.

Connecticut:

Weiss, L.A., 1975, Floodflow formulas for urbanized and non-urbanized areas in Connecticut: in Proceedings of Watershed Management Symposium, American Society of Civil Engineers, Irrigation and Drainage Division, p. 658-675, August 11-13, 1975.

Delaware:

Simmons, R.H., and Carpenter, D.H., 1978, Technique for estimating the magnitude and frequency of floods in Delaware: U.S. Geological Survey Water-Resources Investigations Open-File Report 78-93, 69 p.

Florida:

Seijo, M.A., Giovannelli, R.F., and Turner, J.F., Jr, 1979, Regional flood-frequency relations for west-central Florida: U.S. Geological Survey Open-File Report 79-1293.

Georgia:

Price, McGlone, 1979, Floods in Georgia, magnitude and frequency: U.S. Geological Survey Water-Resources Investigations 78-137 (PB-80 146 244).

Hawaii:

Nakara, R.H., 1980, An analysis of the magnitude and frequency of floods on Oahu, Hawaii: U.S. Geological Survey Water-Resources Investigations 80-45 (PB-81 109 902).

Idaho:

Harenberg, W.A., 1980, Using channel geometry to estimate flood flows at ungaged sites in Idaho: U.S. Geological Survey Water-Resources Investigations 80-32 (PB-81 153 736).

Kjelstrom, L.C., and Moffatt, R.L., 1981, Method of estimating flood-frequency parameters for streams in Idaho: U.S. Geological Survey Open-File Report 81-909.

Thomas, C.A., Harenburg, W.A., and Anderson, J.M., 1973, Magnitude and frequency of floods in small drainage basins in Idaho: U.S. Geological Survey Water-Resources Investigations 7-73 (PB-222 409).

Illinois:

Allen, H.E., Jr., and Bejcek, R.M., 1979, Effects of urbanization on the magnitude and frequency of floods in northeastern Illinois: U.S. Geological Survey Water-Resources Investigations 79-36 (PB-299 065/AS).

Curtis, G.W., 1977, Technique for estimating magnitude and frequency of floods in Illinois: U.S. Geological Survey Water-Resources Investigations 77-117 (PB-277 255/AS).

Indiana:

Davis, L.G., 1974, Floods in Indiana: Technical manual for estimating their magnitude and frequency: U.S. Geological Survey Circular 710.

Gold, R.L., 1980, Flood magnitude and frequency of streams in Indiana--Preliminary estimating equations: U.S. Geological Survey Open-File Report 80-759.

Iowa:

Lara, O.G. 1973, Floods in Iowa: Technical manual for estimating their magnitude and frequency: Iowa Natural Resources Council Bulletin no. 11.

Kansas:

Jordan, P.R., and Irza, T.J., 1975, Magnitude and frequency of floods in Kansas, unregulated streams: Kansas Water Resources Board Technical Report no. 11.

Hedman, E.R., Kastner, W.M., and Hejl, H.R., 1973, Selected streamflow characteristics as related to active-channel geometry of streams in Kansas: Kansas Water Resources Board Technical Report no. 10.

Kentucky:

Hannum, C.H., 1976, Technique for estimating magnitude and frequency of floods in Kentucky: U.S. Geological Survey Water-Resources Investigations 76-62 (PB-263 762/AS).

Louisiana:

Lowe, A.S., 1979, Magnitude and frequency of floods for small watersheds in Louisiana: Louisiana Department of Transportation and Development, Office of Highways, Research Study No. 65-2H.

Neely, B.L., Jr., 1976, Floods in Louisiana, magnitude and frequency, 3d ed., 1976: Louisiana Department of Highways.

Maine:

Morrill, R.A., 1975, A technique for estimating the magnitude and frequency of floods in Maine: U.S. Geological Survey open-file report.

Carpenter, D.H., 1980, Technique for estimating magnitude and frequency of floods in Maryland: U.S. Geological Survey Water-Resources Investigations Open-File Report 80-1016.

Massachusetts:

Wandle, S.W., 1981, Estimating peak discharges of small rural streams in Massachusetts: U.S. Geological Survey Open-File Report 80-676.

Michigan:

Bent, P.C., 1970, A proposed streamflow data program for Michigan: U.S. Geological Survey open-file report.

Minnesota:

Guetzkow, L.C., 1977, Techniques for estimating magnitude and frequency of floods in Minnesota: U.S. Geological Survey Water-Resources Investigations 77-31 (PB-272 509/AS).

Mississippi:

Colson, B.E., and Hudson, J.W., 1976, Flood frequency of Mississippi State Highway Department.

Missouri:

Hauth, L.D., 1974, A technique for estimating the magnitude and frequency of Missouri floods: U.S. Geological Survey open-file report.

Spencer, D.W., and Alexander, T.W., 1978, Technique for estimating the magnitude and frequency of floods in St. Louis County, Missouri: U.S. Geological Survey Water-Resources Investigations 78-139 (PB-298 245/AS).

Montana:

Parrett, Charles, and Omang, R.J., 1981, Revised techniques for estimating magnitude and frequency of floods in Montana: U.S. Geological Survey Open-File Report 81-917.

Nebraska:

Beckman, E.W., 1976, Magnitude and frequency of floods in Nebraska: U.S. Geological Survey Water-Resources Investigations 76-109 (PB-260 842/AS).

Nevada:

Moore, D.O., 1974, Estimating flood discharges in Nevada using channel-geometry measurements: Nevada State Highway Department Hydrologic Report no. 1.

_______, 1976, Estimating peak discharges for small drainages in Nevada according to basin area within elevation zones: Nevada State Highway Department Hydrologic Report no. 3.

New Hampshire:

LeBlanc, D.R., 1978, Progress report on hydrologic investigations of small drainage areas in New Hampshire--Preliminary relations for estimating peak discharges on rural, unregulated streams: U.S. Geological Survey Water-Resources Investigations 78-47 (PB-284 127/AS).

New Jersey:

Stankowski, S.J., 1974, Magnitude and frequency of floods in New Jersey with effects of urbanization: New Jersey Department of Environmental Protection Special Report 38.

New Mexico:

Scott, A.G., 1971, Preliminary flood-frequency relations and summary of maximum discharges in New Mexico--A progress report: U.S. Geological Survey open-file report.

Scott, A.G., and Kunkler, J.L., 1976, Flood discharges of streams in New Mexico as related to channel geometry: U.S. Geological Survey open-file report.

New York:

Zembrzuski, T.J., and Dunn, Bernard, 1979, Techniques for estimating magnitude and frequency of floods on rural unregulated streams in New York excluding Long Island: U.S. Geological Survey Water-Resources Investigations 79-83 (PB-80 201 148).

North Carolina:

Jackson, N.M., Jr., 1976, Magnitude and frequency of floods in North Carolina: U.S. Geological Survey Water-Resources Investigations 76-17 (PB-254 411/AS).

North Dakota:

Crosby, O.A., 1975, Magnitude and frequency of floods in small drainage basins in North Dakota: U.S. Geological Survey Water-Resources Investigations 19-75 (PB-248 480/AS).

Ohio:

Webber, E.E., and Bartlett, W.P., Jr., 1977, Floods in Ohio magnitude and frequency: State of Ohio, Department of Natural Resources, Division of Water, Bulletin 45.

Oklahoma:

Thomas, W.O., Jr., and Carley, R.K., 1977, Techniques for estimating flood discharges for Okahoma streams: U.S. Geological Survey Water-Resources Investigations 77-54 (PB-273 402/AS).

Oregon:

Harris, D.D., Hubbard, L.L., and Hubbard, L.E., 1979, Magnitude and frequency of floods in western Oregon: U.S. Geological Survey Open-File Report 79-553.

Laenen, Antonius, 1980, Storm runoff as related to urbanization in the Portland, Oregon-Vancouver, Washington, area: U.S. Geological Survey Water-Resources Investigations Open-File Report 80-689.

Pennsylvania:

Flippo, H.N., Jr., 1977, Floods in Pennsylvania: A manual for estimation of their magnitude and frequency: Pennsylvania Department of Environmental Resources Bulletin no. 13, 59 p.

Puerto Rico:

Lopez, M.A., Colon-Dieppa, E., and Cobb, E.D., 1978, Floods in Puerto Rico, magnitude and frequency: U.S. Geological Survey Water-Resources Investigations 78-141 (PB-300 855/AS).

Rhode Island:

Johnson, C.G., and Laraway, G.A., 1976, Flood magnitude and frequency of small Rhode Island streams--Preliminary estimating relations: U.S. Geological Survey open-file report.

South Carolina:

Whetstone, B.H., 1982, Floods in South Carolina--Techniques for estimating magnitude and frequency of floods with compilation of flood data: U.S. Geological Survey Water-Resources Investigations 82-1 (78 pages).

South Dakota:

Becker, L.D., 1974, A method for estimating the magnitude and frequency of floods in South Dakota: U.S. Geological Survey Water-Resources Investigations 35-74 (PB-239 831/AS).

_______, 1980, Techniques for estimating flood peaks, volumes, and hydrographs on small streams in South Dakota: U.S. Geological Survey Water-Resources Investigations 80-80 (PB-81 136 145).

Tennessee:

Randolph, W.J., and Gamble, C.R., 1976, Technique for estimating magnitude and frequency of floods in Tennessee: Tennessee Department of Transportation.

Texas:

Dempster, G.R., Jr., 1974, Effects of urbanization on floods in the Dallas, Texas, metropolitan area: U.S. Geological Survey Water-Resources Investigations 60-73 (PB-230 188/AS).

Liscum, Fred, and Massey, B.C., 1980, Technique for estimating the magnitude and frequency of floods in the Houston, Texas, metropolitan area: U.S. Geological Survey Water-Resources Investigations 80-17 (ADA-089 495).

Schroeder, E.E., and Massey, B.C., 1977, Techniques for estimating the magnitude and frequency of floods in Texas: U.S. Geological Survey Water-Resources Investigations Open-File Report 77-110.

Utah:

Butler, Elmer, and Cruff, R.W., 1971, Floods of Utah, magnitude and frequency characteristics through 1969: U.S. Geological Survey open-file report.

Vermont:

Johnson, C.G., and Tasker, G.D., 1974, Flood magnitude and frequency of Vermont streams: U.S. Geological Survey Open-File Report 74-130.

Virginia:

Miller, E.M., 1978, Technique for estimating magnitude and frequency of floods in Virginia: U.S. Geological Survey Water-Resources Investigations Open-File Report 78-5.

Washington:

Cummans, J.E., Collings, M.R., and Nassar, E.G., 1975, Magnitude and frequency of floods in Washington: U.S. Geological Survey Open-File Report 74-336.

West Virginia:

Runner, G.S., 1980, Technique for estimating magnitude and frequency of floods in West Virginia: U.S. Geological Survey Open-File Report 80-1218.

Wisconsin:

Conger, D.H., 1980, Techniques for estimating magnitude and frequency of floods for Wisconsin streams: U.S. Geological Survey Water-Resources Investigations Open-File Report 80-1214.

Wyoming:

Lowham, H.W., 1976, Techniques for estimating flow characteristics of Wyoming streams: U.S. Geological Survey Water-Resources Investigations 76-112 (PB-264 224/AS).

## FEDERAL HIGHWAY ADMINISTRATION

Publications listed below with an (*) are available from the Government Printing Office, Washington, D.C. 20402. The other publications are available in limited numbers to State highway agencies and other public agencies from the Federal Highway Administration. Requests for these documents and suggestions on the contents of any publications should be addressed to the Federal Highway Administration, Office of Engineering, Bridge Division, HNG-31, Washington, D.C. 20590.

### Hydraulic Design Series

| | |
|---|---|
| HDS No. 1 | HYDRAULICS OF BRIDGE WATERWAYS - Second Edition - Revised 1978 |
| HDS No. 3 | DESIGN CHARTS FOR OPEN-CHANNEL FLOW - 1961, Reprinted 1973 |
| HDS No. 4 | DESIGN OF ROADSIDE DRAINAGE CHANNELS - 1965 |

### Hydraulic Engineering Circulars

| | |
|---|---|
| HEC No. 1 | SELECTED BIBLIOGRAPHY OF HYDRAULIC AND HYDROLOGIC SUBJECTS - 1983 |
| HEC No. 3 | HYDROLOGY OF A HIGHWAY STREAM CROSSING - January 1961 |
| HEC No. 5 | HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS - 1965 |
| HEC No. 9 | DEBRIS-CONTROL STRUCTURES - 1971 |
| HEC No. 10 | CAPACITY CHARTS FOR THE HYDRAULIC DESIGN OF HIGHWAY CULVERTS - 1972 |
| HEC No. 11 | USE OF RIPRAP FOR BANK PROTECTION - 1967 |
| * HEC No. 12 | DRAINAGE OF HIGHWAYS PAVEMENTS - 1984 (GPO 050-001-00280-9, $5.50) |
| HEC No. 13 | HYDRAULIC DESIGN OF IMPROVED INLETS FOR CULVERTS - 1972 |
| HEC No. 14 | HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS - 1975 |
| HEC No. 15 | DESIGN OF STABLE CHANNELS WITH FLEXIBLE LININGS - 1975 |
| HEC No. 16 | ADDENDUM TO HIGHWAYS IN THE RIVER ENVIRONMENT - 1980 |
| HEC No. 17 | THE DESIGN OF ENCROACHMENTS ON FLOOD PLAINS USING RISK ANALYSIS - 1980 |

### Electronic Computer Programs

| | |
|---|---|
| HY-2 | HYDRAULIC ANALYSIS OF PIPE-ARCH CULVERTS - 1969 |
| HY-4 | HYDRAULICS OF BRIDGE WATERWAYS - 1969 |
| HY-6 | HYDRAULIC ANALYSIS OF CULVERTS (Box and Circular) - 1979 |

### Calculator Design Series

| | |
|---|---|
| CDS No. 1 | HYDRAULIC DESIGN OF IMPROVED INLETS FOR CULVERTS USING PROGRAMABLE CALCULATORS, (COMPUCORP 325) - October 1980 |
| CDS No. 2 | HYDRAULIC DESIGN OF IMPROVED INLETS FOR CULVERTS USING PROGRAMABLE CALCULATORS, (HP-65) - October 1980 |
| CDS No. 3 | HYDRAULIC DESIGN OF IMPROVED INLETS FOR CULVERTS USING PROGRAMABLE CALCULATORS, (TI-59) - January 1981 |
| CDS No. 4 | HYDRAULIC ANALYSIS OF PIPE-ARCH AND ELLIPTICAL SHAPE CULVERTS USING PROGRAMABLE CALCULATORS, (TI-59) - March 1982 |
| CDS No. 5 | HYDRAULIC DESIGN OF STORMWATER PUMPING STATIONS USING PROGRAMABLE CALCULATORS, (TI-59), May 1982 |