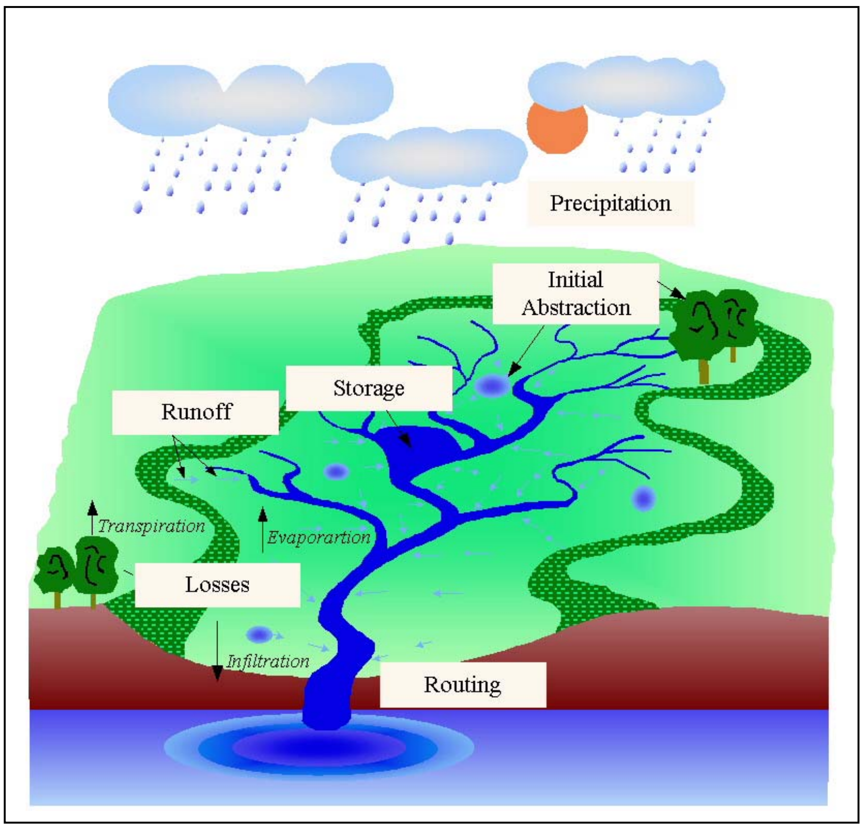
Publication No. FHWA-NHI-02-001
October 2002

U.S. Department of Transportation

Federal Highway Administration

Hydraulic Design Series No. 2, Second Edition

Highway Hydrology



National Highway Institute

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-NHI-02-001 HDS-2		

4. Title and Subtitle	5. Report Date
Highway Hydrology Hydraulic Design Series Number 2, Second Edition	October 2002
	6. Performing Organization Code
7. Author(s) Richard H. McCuen, Peggy A. Johnson, Robert M. Ragan	8. Performing Organization Report No.
9. Performing Organization Name and Address Greenhorne and O'Mara, Inc 9001 Edmonston Road Greenbelt, Maryland 20770	10. Work Unit No. (TRAIS) 11. Contract or Grant No. DTFH61-95-C-00080
12. Sponsoring Agency Name and Address Federal Highway Administration National Highway Institute / Office of Bridge Technology 4600 North Fairfax Drive / 400 Seventh Street Suite 800 / Room 3202 Arlington, Virginia 22203 / Washington D.C. 20590	13. Type of Report and Period Covered Final Report 1st Edition: Oct 1992 – Jun 1996 2nd Edition – Nov 2000 – Nov 2001 14. Sponsoring Agency Code

15. Supplementary Notes
Project Manager: Joe Krolak – FHWA Eastern Resource Center
Technical Editor, second edition: Roger T. Kilgore
Technical Assistance, second edition: Dan Ghere, Paul Koch, Imtiaz Shah, Sung-Hee Kim

16. Abstract
This document discusses the physical processes of the hydrologic cycle that are important to highway engineers. These processes include the approaches, methods and assumptions applied in design and analysis of highway drainage structures.

Hydrologic methods of primary interest are frequency analysis for analyzing rainfall and ungaged data; empirical methods for peak discharge estimation; and hydrograph analysis and synthesis. The document describes the concept and several approaches for determining time of concentration. The peak discharge methods discussed include log Pearson type III, regression equations, the SCS graphical method (curve number method), and rational method. The technical discussion of each peak flow approach also includes urban development applications. The document presents common storage and channel routing techniques related to highway drainage hydrologic analyses. The document describes methods used in the planning and design of stormwater management facilities. Special topics in hydrology include discussions of arid lands hydrology, wetlands hydrology, snowmelt hydrology, and hydrologic modeling, including geographic information system approaches and applications.

This edition includes new sections on wetlands hydrology and snowmelt hydrology, an expanded section on arid lands hydrology, corrections of minor errors, and inclusion of dual units.

17. Key Word	18. Distribution Statement
Hydrology, frequency analysis, peak discharge estimation, urban hydrology, hydrograph development, storage and channel routing, stormwater management, arid lands hydrology, wetland hydrology, snowmelt hydrology, GIS	This document is available to the public from the National technical Information Service, Springfield, Virginia, 22151

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	424	

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ACKNOWLEDGMENTS

First Edition

The first edition is a revision of Hydraulic Engineering Circular No. 19, which was written by Mr. Frank D. Masch. This revision uses portions of the contents of HEC-19 for which we recognize and appreciate the important contributions of Mr. Masch.

Mr. Thomas Krylowski served as the FHWA COTR. Mr. Philip Thompson, Ms. Abbi Ginsberg, and Mr. Arlo Waddoups contributed technical assistance. Dr. Gary A. Lewis, Mr. Wilbert O. Thomas, Jr. (U.S. Geological Survey), and Mr. Lawrence J. Harrison reviewed the draft documents. Ms. Alison R. Montgomery and Ms. Florence Kemerer drafted figures. Ms. Florence Kemerer typed and formatted the text.

Second Edition

This second edition of this document primarily sought to provide calculations in both metric (SI) and conventional English units and add new material on wetlands and snow melt hydrology. Changes in content and format have been introduced where these appeared to be beneficial to readers. Mr. Larry Jones was the FHWA COTR for the second edition. Mr. Philip Thompson of FHWA provided appreciated support and guidance in the preparation of this second edition.

We wish to acknowledge the following individuals at Greenhorne & O'Mara, Inc. for their contributions in the production of this second edition: Ms. Patricia Padgett-Neal, for word processing, Ms. Julie Liptak, for graphic design, and Ms. Deb Daly, for editorial review. Gary Lewis and Bruce Curtis, of Parsons, Inc., provided significant contributions to the new special topics chapter. Thank you very much!

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GLOSSARY

Abstractions: Portions of the total rainfall that do not contribute to direct runoff, including rainfall intercepted by vegetation, rain water stored in depressions, and water that enters the watershed surface and remains beyond the duration of the storm.

Accuracy: The closeness of a statistic or measurements to the true value. It incorporates both bias and precision.

Air convection melt: The portion of snowmelt occurring due to heat transferred from the air above a snow pack to a snowpack.

Albedo: Fraction of incident radiation that is reflected by a surface or body.

Alluvial: Soil and rock material deposited from flowing water.

Analysis: A term that means "to break apart" and that is applied to methods used to break down hydrologic data in order to develop a hydrologic model or design method (see synthesis).

Annual maximum discharge: The largest instantaneous peak discharge in a year.

Antecedent moisture: Water stored in the watershed prior to the start of rainfall.

Attribute File: A computer file that assigns descriptive characteristics to map or georeferenced features. For example, a symbol might be plotted on a computer screen to show the location of a land cover feature. The attribute file would define characteristics such as the land cover type and percent of imperviousness represented by the symbol.

Bankfull discharge rate: The discharge rate when a stream just overflows its natural banks. There is usually no frequency associated with the discharge rate.

Base flow: Stream flow arising from the depletion of ground-water storage.

Basin-development factor: An index of urbanization that accounts for channel improvements, channel lining, storm drains or sewers, and curb-and-guttered streets.

Bias: A systematic error in a statistic or in measurements. A negative bias indicates underpredicition, and a positive bias indicates systematic overprediction.

Binomial distribution: A probability mass function used in hydrologic risk studies. The discrete distribution is based on four assumptions: (1) there are n occurrences, or trials, of the random variable; (2) the n trials are independent; (3) there are only two possible outcomes for each trial; and (4) the probability of the outcomes is constant from trial to trial.

Calibration: The process of deriving optimum values of model coefficients using measured data. Optimality is based on some goodness-of-fit criterion function. Fitting a model using least squares regression is an example of a calibration method.

Celerity: Propagation speed of a flood wave.

Channel routing: Mathematical processes that describe movement and attenuation of unsteady flow (normally a hydrograph) upstream to downstream in a stream channel. Normally used to calculate outflow from a stream channel.

Coefficient of variation: The ratio of the standard deviation to the mean. This dimensionless parameter is abbreviated C_v.

Computer Aided Design (CAD): An automated system to support design, drafting and the display of graphically oriented information.

Confidence Coefficient: A measure of the certainty with which a statement is made. It is often set equal to one minus the level of significance.

Confidence limits: Statistical limits that define an interval in which the true value of a statistic is expected to lie.

Confluence: The location where two rivers join.

Continuity equation: Based on conservation of mass, the continuity equation relates that (for incompressible flow) the discharge rate equals the product of the flow velocity and the cross-sectional area.

Control section: A stream cross-section where the discharge rate is uniquely determined by the depth of flow immediately upstream.

Convolution: The multiplication-translation-addition process used to route a rainfall-excess hyetograph using the unit hydrograph as the routing model.

Correlation coefficient: An index that represents the combined effects of soil characteristics, the land cover, the hydrologic condition, and antecedent soil moisture conditions.

Critical velocity: The velocity where streamflow passed from turbulent to laminar conditions or from laminar to turbulent conditions.

Culvert: An open channel or conduit used primarily to convey flow under highways, railroad embankments, or runways.

Curve number: An index that represents the combined effects of soil characteristics, the land cover, the hydrologic condition, and antecedent soil moisture conditions.

Database: A collection of inter-related data that is stored in a logical collection of files and managed as a unit to serve one or more applications.

Data Plane: A grouping of geographic data such as land cover or soil type that is stored or identified separate from other data.

Dead storage: Storage in a reservoir or detention basin below the elevation of the principal spillway.

Depth-area relationship: A graphical relationship of the ratio of the watershed-averaged rainfall to a point rainfall versus the drainage area. Separate curves are usually given for selected storm durations.

Depth of runoff: An average depth of runoff assumed to be constant over the entire watershed area. Computed as the ratio of the total volume of rainfall excess to the watershed area.

Design flood: A hypothetical flood hydrograph that results from the routing of a design storm rainfall excess and a synthetic unit hydrograph. A return period is usually associated with the design flood, often assumed to be the frequency of the design storm.

Design storm: A hypothetical storm event used in design. It is assumed to represent average or most likely conditions.

Deterministic methods: A class of methods that contain no random components (in contrast to stochastic methods).

Digital Elevation Model (DEM): An array of regularly spaced elevation points in an electronic file.

Digitizer: A device consisting of a table and a cursor with cross hairs (or recticule) that is used to translate the location of map features into digital coordinates.

Dimensionless hydrograph: A hydrograph that has ordinates of the ratio of the discharge to the peak discharge and values on the abscissa of the ratio of time to the time to peak, i.e., q/q_p versus t/t_p.

Direct runoff: The total runoff hydrograph minus base flow.

Drainage density: An index of the concentration of streams in a watershed, as measured by the ratio of the total length of streams to the drainage area.

Energy grade line: The energy state at a channel or conduit section would be the sum of the pressure, velocity, and elevation heads. The energy grade line describes a conceptual link of the energy states between two (or more) channel or conduit locations. The differences in total energy between these two location would be associated with energy losses. The slope of the energy grade line is often refered to as the friction slope.

Envelope curves: Bounds defined approximately by the maximum observed values. The peak discharge envelope curve, which is placed on a graph of peak discharge versus drainage area, is the upper bound of observed peak discharges for any drainage area. The envelope curves are usually established for homogeneous hydrologic regions.

Exceedence probability: The probability that the magnitude of the random variable (e.g., annual maximum flood peak) will be equalled or exceeded in any one time period, often one year.

Evaporation: A net loss of water molecules from a surface.

Field: A character or group of characters that is a component of a record. Each field holds a single data value such as a character representing a land cover type or a group of characters that name a stream.

File: A source from which data can be obtained or a destination to which data can be sent.

Froude number: The ratio of inertia forces to gravity forces, usually expressed as the ratio of the flow velocity to the square root of the product of gravity and a linear dimension (normally depth), i.e., $V/(gL)^{0.5}$. The Froude number is used in the study of fluid motion.

Fusion: The phase conversion of a solid to a liquid.

Generalized skew: A skew value derived by integrating skew values obtained from many sites in a region.

Latent heat: The amount of heat needed to change the phase of a compound with no change in temperature.

Hierarchical File Structure: A data management system structured in the form of a tree. The tree structure minimizes the steps involved when a large number of files, each storing a different type of data, must be traversed to access a specific data item. A field in the first file accessed, analogous to the tree trunk, points to the second file to be accessed, analogous to the correct branch, etc.

Histogram: A graph that shows the frequency of occurrence of a random variable within class intervals as a function of the value of the random variable. The frequency is the ordinate and the value of the random variable is the abscissa, which is divided into class intervals.

Historically adjusted moments: Values of the mean, standard deviation, and skew adjusted using historic flood information.

Hydraulic grade line: The sum of the pressure and elevation heads. Since in an open channel the pressure head can be neglected, the hydraulic grade line is the water surface. This assumption may not be the case in conduit flow.

Hydraulic radius: The ratio of the cross-sectional area of flow to the wetted perimeter.

Hydrograph: A graph of the time distribution of discharge at a point on a stream.

Hydrologic cycle: A representation of the physical processes that control the distribution and movement of water.

Hyetograph: A time-dependent function of the rainfall intensity versus time.

Index-flood method: A peak discharge estimation method that quantifies a peak discharge for a specific exceedence probability by the product of a peak discharge estimated with a regression equation for the index flood and an index ratio.

Infiltration: The process of water entering the upper layers of the soil profile.

Initial abstractions: The portion of the rainfall that occurs prior to the start of direct runoff.

Instantaneous unit hydrograph: The hydrologic response of the watershed to 1-cm of rainfall excess concentrated in an infinitesimally small period of time.

Intensity: Volume per unit time.

Intensity-duration-frequency curve: A graph or mathematical equation that relates the rainfall intensity, storm duration, and exceedence frequency.

Isohyet: A line on a map of equal rainfall depth for the same duration, usually the duration of a storm.

Land cover/land use: Most conventional definitions have land cover relating to the type of feature on the surface of the earth such as rooftop, asphalt surface, grass and trees. Land use associates the cover with a socio-economic activity such as factory or school, parking lot or highway, golf course or pasture and orchard or forest. Hydrologic modeling often uses the terms land cover and land use interchanagably because the inputs to the models require elements from each definition.

Latent heat of fusion: Heat necessary to change ice to water (for ice at 0^oC, it is 79.7 calories per gram).

Latent heat of sublimation: Heat necessary to change ice to vapor (for ice at 0^oC, it is 676 calories per gram).

Latent heat of vaporization: Heat necessary to change liquid water to vapor (for water at 0^oC, it is 596 calories per gram).

Least squares regression: A procedure for fitting a mathematical function that minimizes the sum of the squares of the differences between the predicted and measured values.

Level of significance: A statistical concept that equals the probability of making a specific error, namely of rejecting the null hypothesis when, in fact, it is true. The level of significance is used in statistical decision-making.

Maximum likelihood estimation: A mathematical method of obtaining the parameters of a probability distribution by optimizing a likelihood function that yields the most likely parameters based on the sample information.

Method-of-moments estimation: A method of fitting the parameters of a probability distribution by equating them to the sample moments.

Moving-average smoothing: A statistical method of smoothing a time or space series in which the nonsystematic variation is eliminated by averaging adjacent measurements. The smoothed series represents the systematic variation.

Nonhomogeneity: A characteristic of time or space series that indicates the moments are not constant throughout the length of the series.

Nonparametric statistics: A class of statistical tests that do not require assumptions about the population distribution.

Order-theory statistics: A class of statistical methods in which the analysis is based primarily on the order relations among the sample values.

Orifice equation: An equation based on Bernoulli's equation that relates the discharge through an orifice to the area of the orifice and the depth of water above the center of the orifice.

Outlier: An extreme event in a data sample that has been proven using statistical methods to be from a population different from the remainder of the data.

Parametric statistics: A class of statistical tests in which their derivation involved explicit assumptions about the underlying population.

Partial-duration frequency analysis: A frequency method that uses all floods of record above a threshold to derive a probability function to represent the data.

Pearson correlation coefficient: An index of association between paired values of two random variables. The value assumes a linear model.

Physically-based Hydrologic Models: That family of models that estimate runoff by simulating the behavior and watershed linkages of individual processes such as infiltration, depression and detention storage, overland and channel flows, etc.

Pixel: An array of picture elements on a color screen of a personal computer.

Plotting position formula: An equation used in frequency analysis to compute the probability of an event based on the rank of the event and the sample size.

Power model: A mathematical function that relates the criterion (dependent) variable, y, to the predictor (independent) variable, x, raised to an exponent, i.e., $y = ax^b$.

Precision: A measure of the nonsystematic variation. It is the ability of an estimator to give repeated estimates that are close together.

Probability paper: A graph paper in which the ordinate is the value of a random variable and the abscissa is the probability of the value of the random variable being equaled or exceeded. The nature of the probability scale depends on the probability distribution.

Radiation Melt: The portion of snowmelt occurring due to solar radiation providing energy to a snowpack.

Rainfall excess: The portion of rainfall that causes direct flood runoff. It equals the total rainfall minus the initial abstraction and losses.

Random Access: Access to stored data in which the data can be referred to in any order whatever, instead of just in the order in which they are stored.

Raster Database: A method for displaying and storing geographic data as a rectangular array of characters where each character represents the dominant feature, such as a land cover or soil type, in a grid cell at the corresponding location on a map.

Rating curve: A graph or mathematical equation that relates the stage (h) and discharge (q). Often expressed with a power model form, $q = ah^b$.

Real-time modeling: Hydrologic modeling in which a calibrated model is used with data for a storm event in progress to make predictions of streamflow for the remainder of the storm event.

Record: A string of characters or groups of characters (fields) that are treated as a single unit in a file.

Representative Channel Cross-Section: A cross-section that is selected for use in a model because the flow characteristics through that section are considered to be typical or representative of the flow conditions along a given length of a river or stream.

Reservoir routing: Mathematical relations used to calculate outflow from a reservoir.

Return period: A concept used to define the average length of time between occurrences in which the value of the random variable is equaled or exceeded.

Risk: The probability that an event of a given magnitude will be equaled or exceeded within a specific period of time.

Scanner: A device that measures the light passing through or the reflectance of light from a map or other document to convert the data into a computer compatible raster format file. Subsequent operations can then translate the raster data into vector formats, land cover files, etc.

SCS County Soil Map: A book prepared by the Soil Conservation Service of the USDA that describes and discusses the soil related environment and presents maps showing the distribution of soil characteristics for a county.

S-hydrograph: The cumulative hydrograph that results from adding an infinite number of T-hour unit hydrographs, each lagged T-hours.

Sheet flow: Shallow flow on the watershed surface that occurs prior to the flow concentrating into rills.

Skew: The third statistical moment, with the mean and variance being the first and second statistical moments. The skew is a measure of the symmetry of either data or a population distribution, with a value of zero indicating a symmetric distribution.

Slope-area method: A method of estimating discharge rates using basic equations of hydraulics, such as Manning's equation and the continuity equation.

Snow Water Equivalent (SWE): A resulting depth of water obtained by melting the snow from a given snow event. Units are usually expressed in millimeters (inches) of water.

Spearman correlation coefficient: An index of association between paired values of two random variables. It is computed using the ranks of the data rather than the sample values. It is the nonparametric alternative to the Pearson correlation coefficient.

Specific energy: The total energy head measured above the channel bed at a specific section of channel. Calculated as the sum of the velocity head and the depth of flow. The minimum specific energy occurs at critical depth.

Specific heat: The amount of heat necessary to raise the temperature of a compound over a given temperature interval without a change in state.

Stage-storage-discharge relationships: A relationship between stage, storage, and discharge used in storage routing methods. It is usually computed from the stage-storage and stage-discharge relationships.

Standard error: A measure of the sampling variation of a statistic.

Standard error of estimate: The standard deviation of the residuals in a regression analysis. It is based on the number of degrees of freedom associated with the errors.

Sublimation: The phase conversion of a solid to a gas.

Synthesis: The term means "To put together" and is applied to the problem of hydrologic estimation using a known model (see analysis).

Synthetic unit hydrograph: A unit hydrograph not directly based on measured rainfall and runoff data.

TIGER/Line Files: Topologically Integrated Geographically Encoding and Referencing (TIGER) system available on CD-ROM from the U.S. Bureau of Census. The files store vector segments that when connected form line features such as streets and streams. The files also provide the names of the individual streets and streams and the street addresses between intersections.

Time-area curve: The relationship between runoff travel time and the portion of the watershed that contributes runoff during that travel time.

Time of concentration: The time required for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point.

Unit peak discharge: The peak discharge per unit area, with units of $m^3/sec/km^2$.

Vaporization: The phase conversion of a liquid to a gas.

Vapor condensation melt: The portion of snowmelt occurring due to heat released by water vapor as it condenses on the snowpack and converts to liquid water.

Vector Database: A method for displaying and storing geographic data as a distribution of vector segments that, when connected, form polygons that enclose homogeneous areas such as a defined land cover or form lines representing features such as roads or streams.

Water year: October 1 to September 30, with the water year number taken as the calendar year of the January 1 to September 30 period.

Weighted skew: An estimate of the skew based on both the station skew and a regionalized value of skew.

Wetted perimeter: The length of a cross section normal to flow in which the water is in contact with the stream bed or banks.

Work Station: A combination of hardware and software normally used by one person to interact with a computer system and perform computer supported tasks.

Zero-flood records: Annual maximum flood records that include zero values or values below a threshold.

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CHAPTER 1

INTRODUCTION

Hydrology is often defined as the science that addresses 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 as there are individual bodies of science dedicated to the study of various elements contained within this definition. Meteorology, oceanography, and geohydrology, among others, are typical. For the highway designer, the primary focus of hydrology is the water that moves on the earth's surface and in particular that part that ultimately crosses transportation arterials (i.e., highway stream crossings). A secondary interest is to provide interior drainage for roadways, median areas, and interchanges.

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. Hydrologists have defined most of the factors and parameters that influence surface runoff. However, for many of these surface runoff factors, complete functional descriptions of their individual effects exist only in empirical form. Their qualitative analysis requires extensive field data, empirically determined coefficients, and sound judgment and experience.

By application of the principles and methods of modern hydrology, it is possible to obtain solutions that 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 techniques 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. Water 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 from 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 1.1, is a description of the transformation of water from one phase to another and its motion from one location to another. In this context, it represents the complete descriptive 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 level at which condensation occurs, precipitation in the form of rain, hail, sleet, or snow forms and then falls on a watershed. Some of the water evaporates as it falls 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 reaches the ground's surface or onto the water surfaces of rivers, lakes, ponds, and oceans.

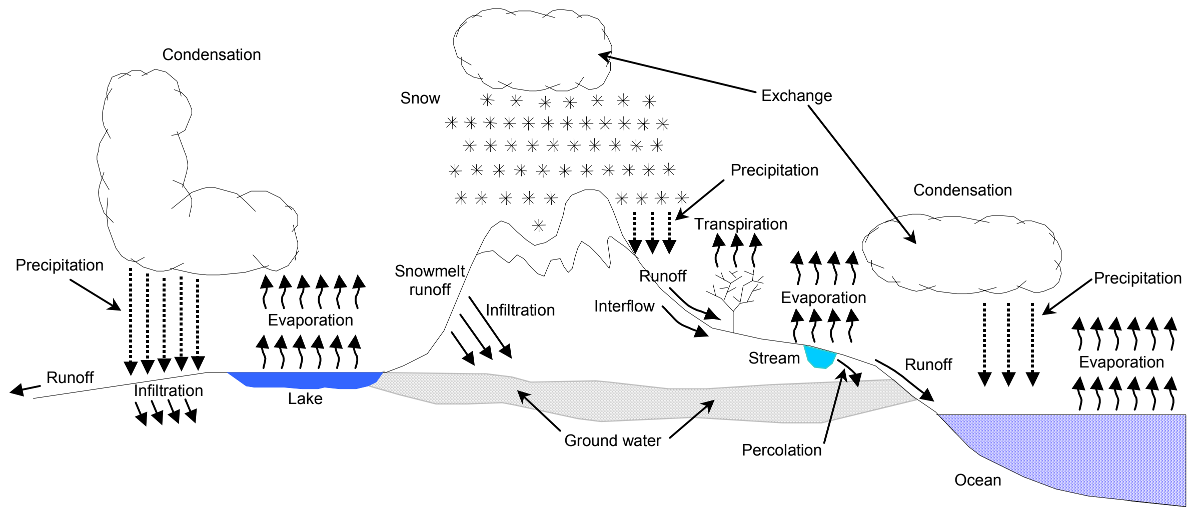

Figure 1.1. The hydrologic cycle

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 occurs. While contained in a snowpack, some of the water escapes 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 evaporates, infiltrates into the root zone, or flows overland into puddles and depressions in the ground or into swales and streams. The effect of infiltration is to increase the soil moisture. Field capacity is the moisture held by the soil after all gravitational drainage. 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.

The part of precipitation that 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 systems, the flow path must reach its storage capacity. This form of storage, called detention storage, is temporary since most of this water continues to drain after 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 reservoirs 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 pavements, bridges, culverts, siphons, and other cross drainage structures for channels varying from small streams to large rivers. Stable open channels and stormwater collection, conveyance, and detention systems must be designed for both urban and rural areas. It is often necessary to evaluate the impacts that future land use, proposed flood control and water supply projects, and other planned and projected changes will have 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 Stream 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. The four most important parts of the hydrologic cycle to the highway designer are: (1) precipitation, (2) infiltration, (3) storage, and (4) surface runoff. Runoff processes are discussed in Chapter 2.

Precipitation is very important to the development of hydrographs and especially in synthetic unit hydrograph methods and some peak discharge formulas where the flood flow is determined in part from excess rainfall or total precipitation minus the sum of the infiltration and storage. As described above, infiltration is that portion of the rainfall that enters the ground surface to become ground water 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 ground-water 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 water stored in more significant quantities often in human-made structures (detention storage). Surface runoff is the water that 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 approximations of these factors. 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 Overview of Hydrology as Applied to Stream Crossings

In many hydrologic analyses, the three basic elements are: (1) measurement, recording, compilation, and publication of data; (2) interpretation and analysis of data; and (3) application to design or other practical problems.

The development of hydrology for a highway stream crossing is no different. Each of these tasks must be performed, at least in part, before an actual hydraulic structure can be designed. How extensively involved the designer becomes with each depends on: (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; and (5) time and other resource constraints.

These factors normally determine the level of analysis needed and justified for any particular design situation. As practicing designers will confirm, they may be 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 that are difficult to acquire. The more demanding methods and techniques should be reserved for those special projects where additional data collection and accuracy produce benefits that 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 HEC-1 or HEC-HMS, developed by the U.S. Army Corps of Engineers (USACE), and TR-20, developed by the Soil Conservation Service (SCS). (The SCS has been renamed the Natural Resources Conservation Service (NRCS)).

There are, however, a number of simpler but equally sound and proven methods available to analyze the hydrology for some common design problems. These procedures enable peak flows and hydrographs to be determined without an excessive expenditure of time and that use existing data or, in the absence of data, synthesize 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 an 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. Peak discharge (sometimes called the instantaneous maximum discharge) is critical because most highway stream crossings 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 that is defined based on a predictable frequency of occurrence (i.e., a 10-year flood, a 50-year flood, etc.). Table 1.1 provides examples of some typical design frequencies for hydraulic structures associated with different roadway classifications, as identified in drainage guidance developed by the American Association of State Highway and Transportation Officials (AASHTO) (AASHTO, 1999).

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 that 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 rates and volumes is required. Similarly, 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.

Table 1.1. Design Storm Selection Guidelines (AASHTO, 1999)

Roadway Classification	Exceedence Probability	Return Period
Rural Principal Arterial System	2%	50-year
Rural Minor Arterial System	4% - 2%	25-50-year
Rural Collector System, Major	4%	25-year
Rural Collector System, Minor	10%	10-year
Rural Local Road System	20% - 10%	5-10-year
Urban Principal Arterial System	4% - 2%	25-50-year
Urban Minor Arterial Street System	4%	25-year
Urban Collector Street System	10%	10-year
Urban Local Street System	20% - 10%	5-10-year

Note: Federal regulations require interstate highways to be provided with protection from the 2 percent flood event. AASHTO recommends that facilities such as underpasses, depressed roadways, etc., where no overflow relief is available should also be designed for the 2 percent flood event (AASHTO, 1999).

Neither peak flow nor hydrographs present any real computational difficulties provided data are available for their determination. A problem faced by the highway designer is that insufficient flow data, or often no data, exist at the site where a stream crossing is to be designed. Although 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 parameters. These methods require considerably more judgment 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 stream flow data for a watershed that 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 gauge datum, moving of a gauge, or mixed floods (floods caused by rainfall and snowmelt or rainfall and hurricanes) must be carefully analyzed to avoid misinterpretation and/or incorrect conclusions.

1.2.3 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, these channelized areas usually have an increase in the peak discharge and a decrease in the time to peak of the runoff hydrograph.

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 must be able to evaluate the effects of channelization work done by others on highway design as well as any channel modifications made in conjunction with highway construction.

1.2.4 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 Chapter 7 can be used to make assessments of these quantities.

1.2.5 Diversions and Dam Construction

The highway designer needs to be aware of the construction or planned construction of diversions or dams on the watershed because these works will significantly affect the magnitude and character of the runoff reaching the highway crossing. The designer should make a point to become informed of proposed projects being studied by the various water resources agencies active in their part of the country. 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 can be used to assess the effects that such projects will have on highway drainage. Recently, the practice of decommissioning dams has increased. Effects on drainage of highways downstream need to be considered.

1.2.6 Natural Disasters

Highways are considered 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 that 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). Chapter 4 states 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 the design life of a structure, 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 Chapter 4 (risk assessment).

Checking for the effects of a rare event is one method of focusing the designer's 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 and consider secondary outlets for the flows. It is very difficult to assign a recurrence interval to such natural disasters, but their impacts need to be assessed.

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 (NWS) and documentation by the NWS is available for consultation and guidance.

After a natural disaster strikes, detailed studies of the effects may be made and reports generated that can serve as guidance to the designer. The NWS, the U.S. Geological Survey (USGS), and the Corps of Engineers are the primary sources of such reports. The primary responsibility for disaster recovery within the Federal Government rests with the Federal Emergency Management Agency (FEMA).

1.3 GENERAL DATA REQUIREMENTS

Regardless of the method selected for the analysis of a particular hydrologic problem, there is a need for data or analysis methods that are based on statistical manipulation of 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 data will be determined in part by the method selected for the design.

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 physical aspects of the rainfall-runoff process while statistical methods utilize measured data to fit functions that represent the process. Deterministic methods can either be conceptual, where each element 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. For example, unit hydrograph methods are deterministic. Statistical methods apply the techniques and procedures of modern statistical analysis to actual or synthetic data and fit the needed design parameters directly. Flood frequency analysis and peak-discharge regression equations are examples of the statistical approach.

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 approach used, and it is not uncommon for two different designers to arrive at 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 that 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. Unit hydrograph methods such as the SCS TR-20 program and the Corps of Engineers HEC-1 program are deterministic methods. Hydrologic channel routing methods such as the Muskingum method are deterministic.

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 that are applied to measured or observed data. The predictions of one designer should be very nearly the same as those of another who applies the same procedures with the same data. The accuracy of statistical methods can also be measured quantitatively. However, statistical methods may not be well understood and, as a result, answers may be misinterpreted. To provide clear guidance, this manual presents the commonly accepted statistical methods for peak flow determination in a logical format that is compatible with the practical needs of highway drainage design.

1.5 ANALYSIS VERSUS SYNTHESIS

Like most of the basic sciences, hydrology requires both analysis and synthesis to use fundamental concepts in the solution of engineering problems. The word analysis is derived from the Greek word analusis, which means "a releasing," and from analuein, which means "to undo." In practical terms, it means "to break apart" or "to separate into its fundamental constituents." Analysis should be compared with the word synthesis. The word synthesis comes from the Latin word suntithenai, which means "to put together." In practical terms, it means "to combine separate elements to form a whole." The meanings of the words analysis and synthesis given here may differ from common usage. Specifically, practicing engineers often use analysis as a synonym for design. This difference needs to be recognized and understood.

Because of the complexity of many hydrologic engineering problems, the fundamental elements of the hydrologic sciences cannot be used directly. Instead, it is necessary to take measurements of the response of a hydrologic process and analyze the measurements in an attempt to understand how the process functions. Quite frequently, a model is formulated on the basis of the physical concepts that underlie the process and the fitting of the hydrologic model with the measurements provides the basis for understanding how the physical process varies as the input to the process varies. After the measurements have been analyzed (i.e., taken apart) to fit the model, the model can be used to synthesize (i.e., put together) design rules. That is, the analysis leads to a set of systematic rules that explain how the underlying hydrologic processes will function in the future. The act of synthesizing is not, of course, a total reproduction of the original process. It is a simplification. As with any simplification, it will not provide a totally precise representation of the physical process under all conditions. But, in general, it should provide reasonable solutions, especially when many designs based on the same design rules are considered.

It should be emphasized that almost every hydrologic design (or synthesis) was preceded by a hydrologic analysis. Most often, one hydrologic analysis is used as the basis for many, many hydrologic designs. But the important point is that the designer must understand the basis for the analysis that underlies any design method; otherwise, the designer may not apply the design procedure in a way that is compatible with the underlying analysis. This is not to say that a design method cannot be applied without knowing the underlying basis, only that it is best when the design engineer fully understands the analysis that led to development of the design rules. Anyone can substitute the values of input variables into a design method. But when a design is used under circumstances for which it was not intended, inaccurate designs could be the result.

Hydrologic models are commonly used without the user taking the time to determine the analysis that underlies the model. In cases where the user is fortunate enough to be applying the model within the proper bounds of the analysis, the accuracy of the design is probably within the limits established by the analysis; however, inaccurate designs can result because the assumptions used in the analysis are not valid for the particular design. Those involved in the analysis phase should clearly define the limits of the model, and those involved in synthesis, or design, should make sure that the design does not require using the model outside the bounds established by the analysis.

1.5.1 A Conceptual Representation of Analysis and Synthesis

Because of the importance of the concepts of analysis and synthesis, it may be worthwhile to place the design problem in a conceptual hydrologic system of three parts: the input, the output, and the transfer function. This conceptual framework is shown schematically in Figure 1.2. In the analysis phase, the input and output are known and the analyst must find a rational model of the transfer function. When the analysis phase is completed, either the model of the transfer function or design tools developed from the model are ready to be used in the synthesis phase. In the synthesis or design phase, the design input and the model of the transfer function are known and the predicted system output must be computed; the true system output is unknown. The designer predicts the response of the system using the model and bases the engineering design solution on the predicted or synthesized response.

1.5.2 Examples of Analysis and Synthesis in Hydrologic Design

Two hydrologic design methods available to the highway engineer are peak-discharge regression equations and unit hydrograph models. These can be used to illustrate factors that must be considered in analysis and synthesis.

Peak-discharge regression equations are commonly used for the design of a variety of highway facilities, such as bridges, culverts, and roadway inlets. In the analysis phase, the input consists of values of watershed characteristics at gauged stations in a homogeneous region. The output is the peak discharge values for a selected return period from frequency analyses at gauged locations. The transfer function, or model, is the power model with unknown regression coefficients. Least-squares regression analyses usually use the watershed characteristics and peak discharge magnitudes from the known watersheds to fit the unknown coefficients. Important assumptions are made in this phase of modeling. Although these assumptions may limit the use of the equations, they are necessary. Specifically, only gauged data from unregulated streams are used. Additionally, stream records used in the frequency analyses should not include watersheds that have undergone extensive watershed change, such as urbanization or deforestation, unless this is specifically accounted for in the flood frequency analyses. Each of the watershed characteristics applies to certain ranges; for example, the drainage areas included in the analyses may range from 50 to 200 square kilometers (20 to 80 square miles). These limits are important to know so that the model is not used without caution beyond the ranges of the inputs used to fit the equation. Failure to understand these factors can lead to an inappropriate use of the fitted model.

It is important to know the accuracy that can be expected of a model, which might be indicated by a standard error of estimate or correlation coefficient of the fitted model. This is important if the engineer wants to compare alternative models when selecting a design method and when the engineer is considering the accuracy of the design. This is also important if the designer wants to compare alternative models when selecting a design method and when considering the accuracy of a design.

In the synthesis phase, the fitted model and values of watershed characteristics at an ungauged location are available; these represent the transfer function and input, respectively. The output is the computed discharge estimate. There is no direct way to assess the accuracy of the design estimate, so the accuracy statistics of the fitted equation are used as estimates of the accuracy of the computed value.

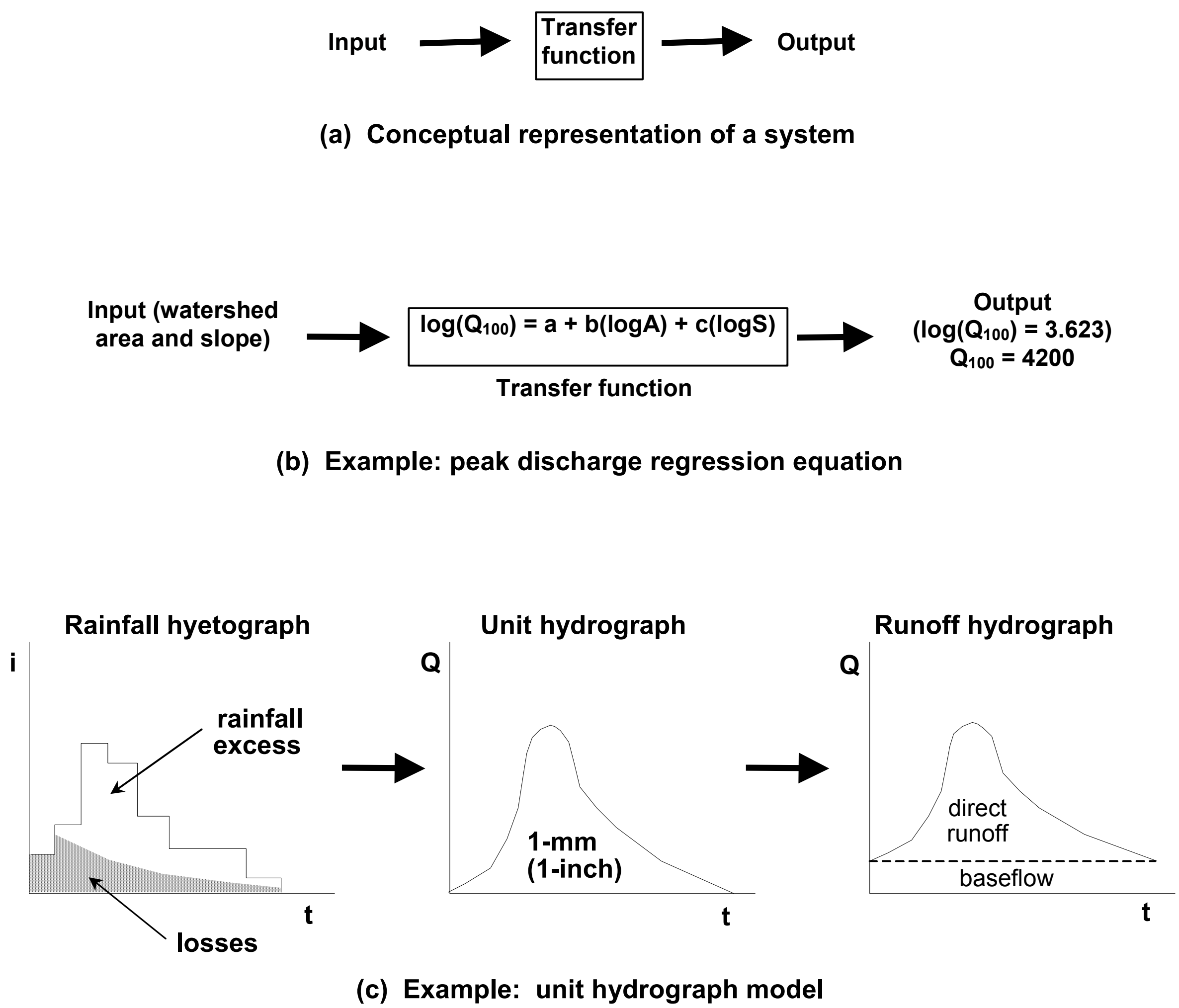

Figure 1.2. Concept and examples of the systems analysis and synthesis process

Unit hydrograph models, which are introduced in Chapter 6, can be used for design work where either the watershed is not homogeneous or storage is a significant factor. To develop a unit hydrograph, which is the transfer function, both a measured rainfall hyetograph and the storm hydrograph measured for the same storm event are needed. The hyetograph is the input function and the hydrograph is the output function. When possible, hyetographs and hydrographs for several storm events should be available to fit unit hydrographs. Then the individual unit hydrographs can be averaged to obtain a more representative unit hydrograph.

An engineer who uses the unit hydrograph for design work should know factors such as the size and character of the watersheds from which it was developed. A unit hydrograph based on data from a coastal area may lead to underdesign if it is used on a mountainous watershed. If the fitted unit hydrograph does not provide an accurate reproduction of the outflow hydrographs used in its development, it will not be reliable and should be used with caution.

In the synthesis phase, the unit hydrograph, as the transfer function, is used with a design storm, which is the input function. The design hydrograph obtained by convolving the design storm and unit hydrograph is the output function. The accuracy of the design hydrograph will depend on the accuracy of the unit hydrograph and its appropriateness for the watershed for which the design is being made.

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CHAPTER 2

RAINFALL/RUNOFF PROCESSES

From the discussion of the hydrologic cycle in Chapter 1, 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 portion of the total precipitation that has not been removed by processes in the hydrologic cycle. The amount of precipitation that runs off from the watershed is called the "rainfall excess", and "hydrologic abstractions" is the commonly used term that groups all of the processes that extract water from the original precipitation. It follows then that the volume of surface runoff is equal to the volume of 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 chapter is to describe more fully the runoff process. An understanding of the process is necessary to properly apply hydrologic design methods. 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 chapter includes a qualitative discussion of the runoff process, beginning with precipitation and illustrating how this input is modified by each of the hydrologic abstractions. Because the time characteristics of runoff are important in design, a discussion of runoff travel time parameters is included.

2.1 PRECIPITATION

Precipitation is the water that falls from the atmosphere in either liquid or solid form. It results from the condensation of moisture in the atmosphere due to the 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 that 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 melting occurs. Much of the precipitation that falls in mountainous areas and in the northerly latitudes falls in the 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 that causes the precipitation. Each type is characterized by different spatial and temporal rainfall regimens. The three major types of storms are classified as convective storms, orographic storms, and 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 2.1. The characteristic form of convective precipitation is the summer thunderstorm. The 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 and depths are quite common beneath slowly moving thunderstorms.

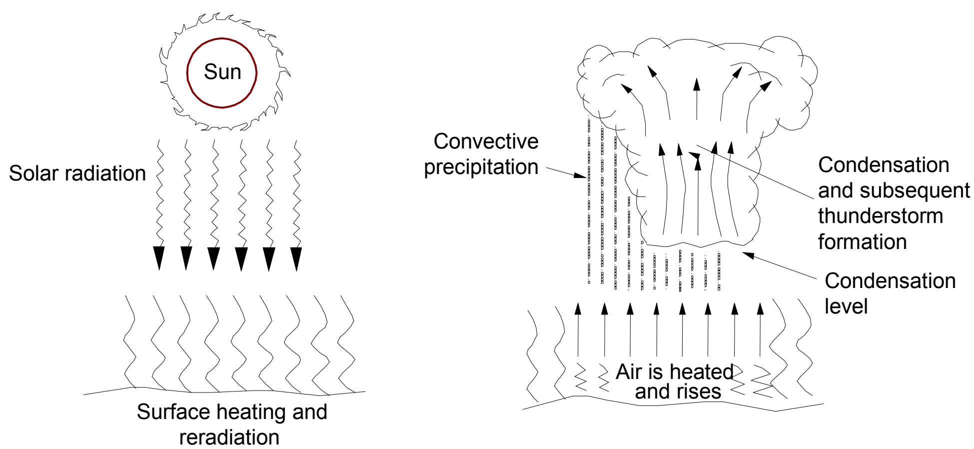

Figure 2.1. Convective storm

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 (see Figure 2.2). 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 2500 mm (100 in) of precipitation annually, while the east-facing slopes, only a short distance away over the crest of the mountains, receive on the order of 500 mm (20 in) of precipitation annually.

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.

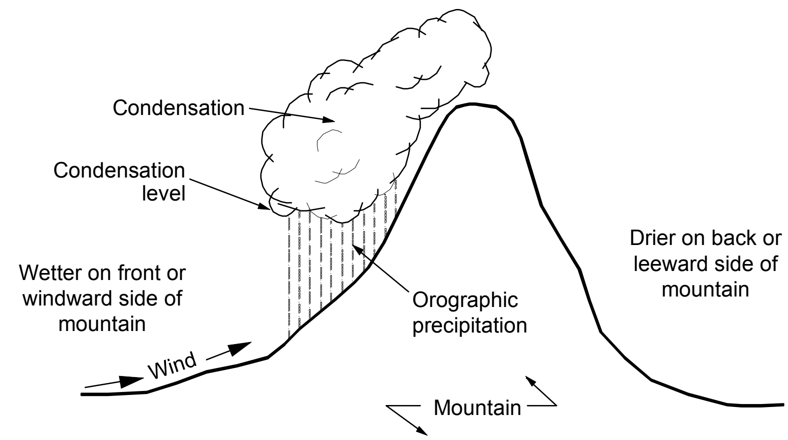

Figure 2.2. Orographic storm

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 2.3, 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 (see Figure 2.4). 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 and Typhoons

Hurricanes, typhoons, or tropical cyclones develop over tropical oceans that have a surface-water temperature greater than 29°C (84°F). 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 350 to 500 mm (14 to 20 in) in less than 24 hours are common in well-developed hurricanes, where winds are often sustained in excess of 120 km/h (75 mi/h).

2.1.3 Characteristics of Rainfall Events

The characteristics of precipitation that are important to highway drainage are the intensity (rate of rainfall); the duration; the time distribution of rainfall; the storm shape, size, and movement; and the frequency.

Intensity is defined as the time rate of rainfall depth and is commonly given in the units of millimeters per hour (inches per hour). All precipitation is measured as the vertical depth of water (or water equivalent in the case of snow) that would accumulate on a flat level surface if all the precipitation remained where it fell. A variety of rain gauges have been devised to measure precipitation. All first-order weather stations use gauges that provide nearly continuous records of accumulated rainfall with time. These data are typically reported in either tabular form or as cumulative mass rainfall curves (see Figure 2.5).

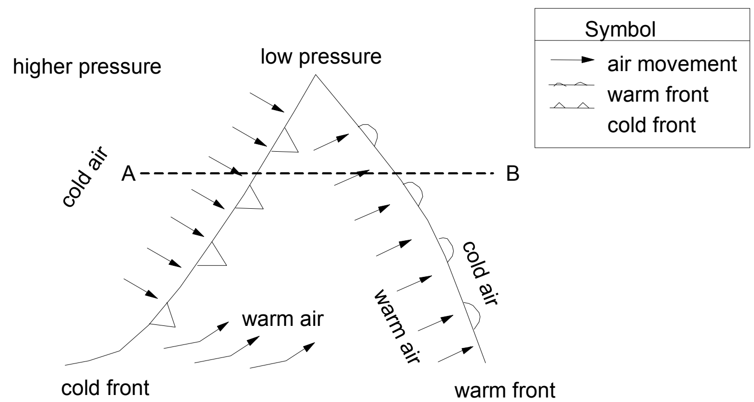

Figure 2.3. Storm as it appears on weather map in the northern hemisphere

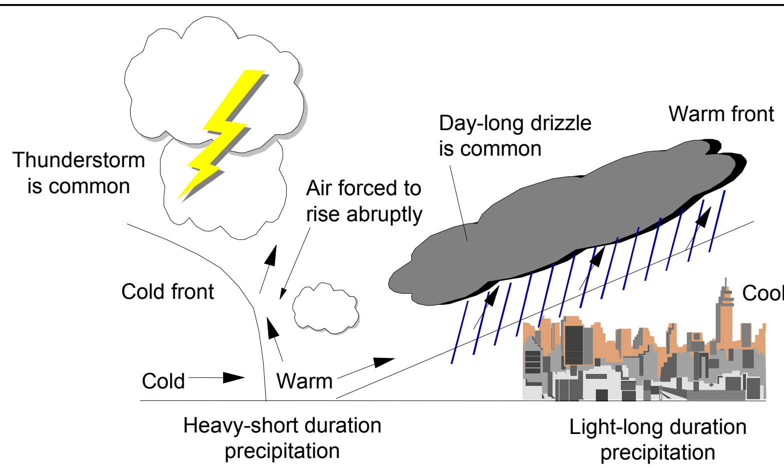

Figure 2.4. Cyclonic storms in mid-latitude; cross-section from A to B of Figure 2.3

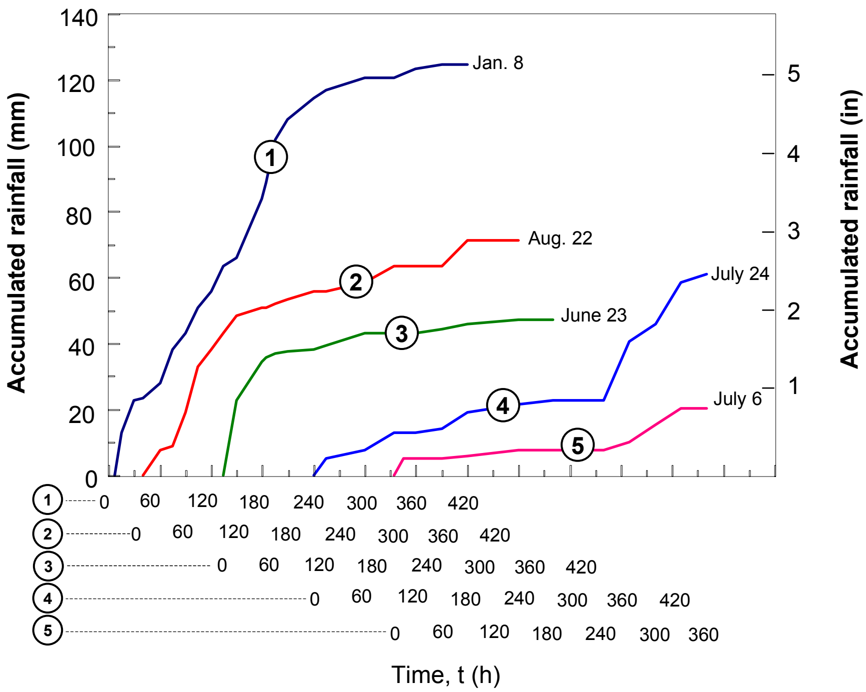

Figure 2.5. Typical mass rainfall curves: Kickapoo Station recording gauge

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 in Figure 2.6 for the Kickapoo Station.

While the above illustrations use a 1-hour time 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 2.6 shows the irregular and complex nature of different storms measured at the same station.

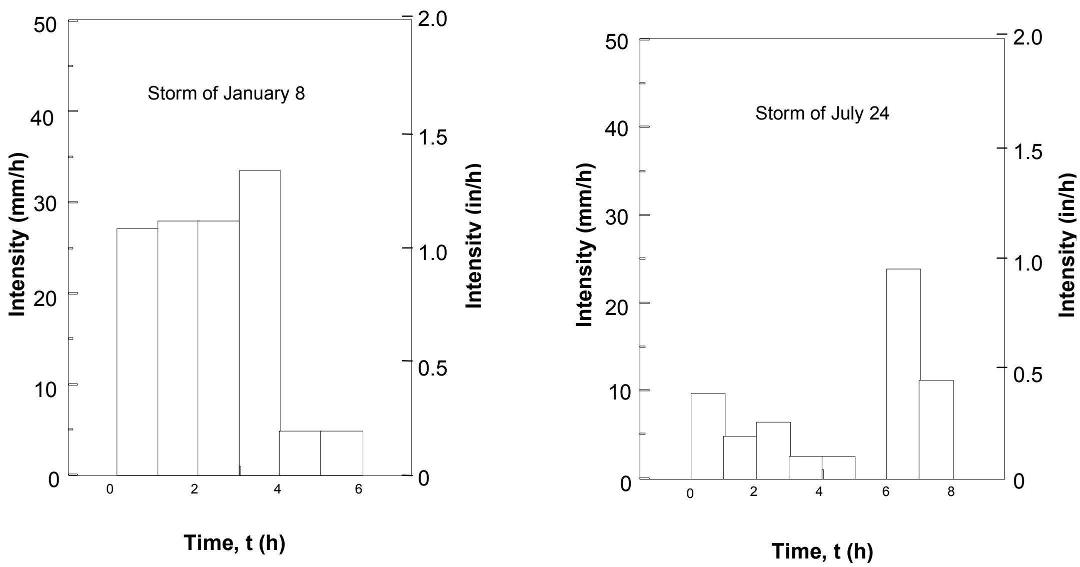

Figure 2.6. Rainfall hyetographs for Kickapoo Station

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 rate from a given watershed. Intensities can vary from misting conditions where a trace of precipitation may fall to cloudbursts. Figure 2.7 summarizes some of the maximum observed rainfalls in the United States. The events given in Figure 2.7 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 150 to 500 mm/h (6 to 20 in/h) for the shorter durations (<1 hour) and from 50 to 250 mm/h (2 to 10 in/h) 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 2.5 or 2.6. In the case of Figure 2.5, 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 2.6, 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 2.6. This time variation directly determines the corresponding distribution of the surface runoff. As illustrated in Figure 2.8, high intensity rainfall at the beginning of a storm, such as the January 8 storm in Figure 2.6, will usually 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 2.6, the time to peak will be longer, followed by a rapidly falling recession.

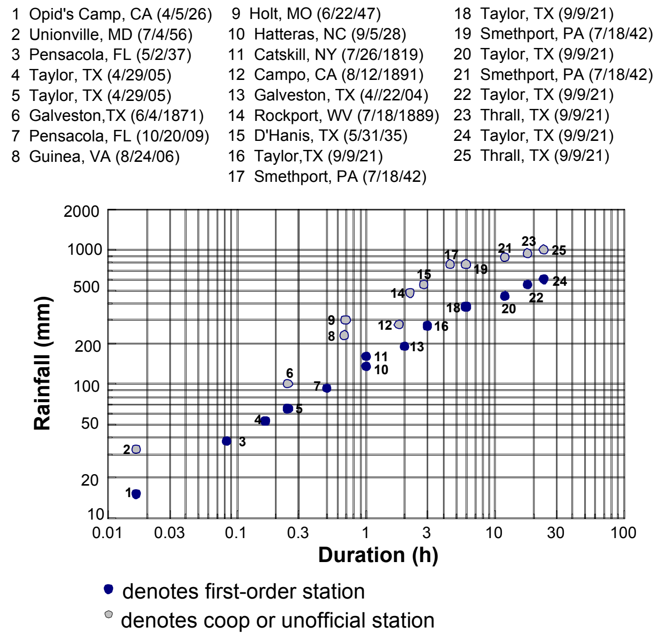

Figure 2.7. Maximum observed rainfalls (U.S.) from USWB, 1947; ECAFE, U.N., 1967

Storm pattern, areal extent, and movement are normally determined by the type of storm (see Section 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 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 2.9, a small localized storm of a given intensity and duration, occurring over a part of the drainage area, will result in much less runoff than if the same storm covered the entire watershed.

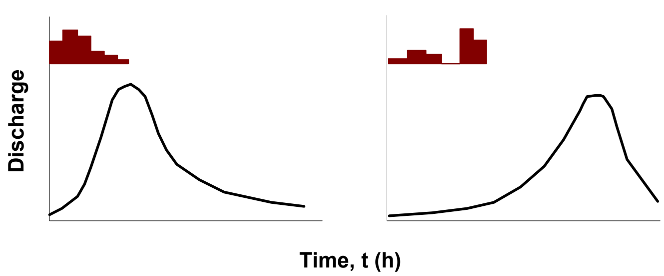

Figure 2.8. Effect of time variation of rainfall intensity on the surface runoff

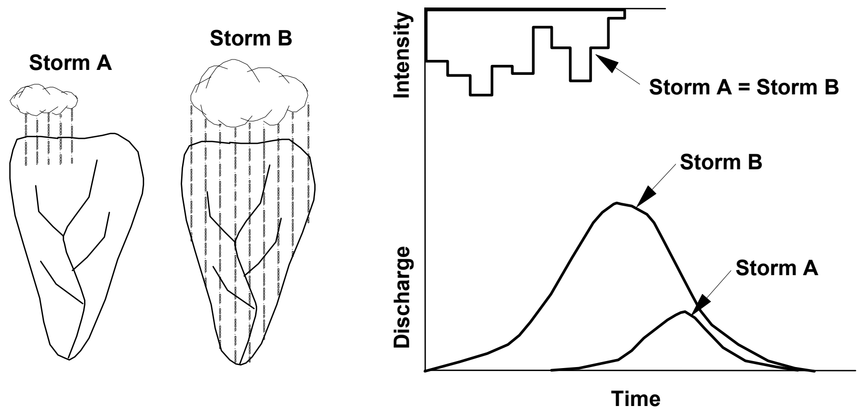

Figure 2.9. Effect of storm size on runoff hydrograph

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 occurred in a remote part of the basin, the runoff at the outlet due to the storm would be longer and the peak flow lower due to storage in the channel.

Storm movement has a similar effect on the runoff distribution particularly if the basin is long and narrow. Figure 2.10 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.

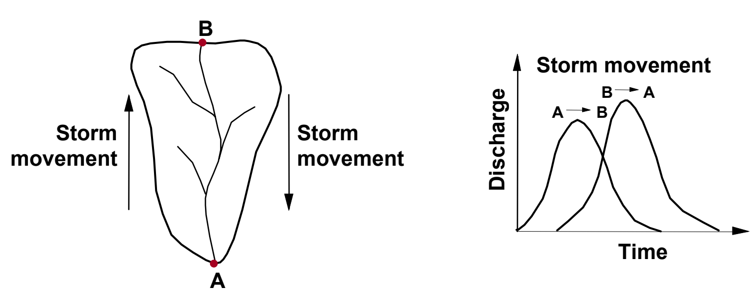

Figure 2.10. Effect of storm movement on runoff hydrograph

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 storm of a given frequency always produces a flood of the same frequency, there are a number of analytical techniques that are based on this assumption, particularly for ungauged watersheds. Some of the factors that determine how closely the frequencies of precipitation and peak discharge correlate with one another are discussed further in Section 2.3.

Precipitation is not easily characterized although there have been many attempts to do so. References and data sources are available that provide general information on the character of precipitation at specified geographic locations. These sources are discussed more fully in Chapter 3. 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 that are indigenous to their regions of concern. They should also understand the seasonal variations that are prevalent in many areas. In addition, it is very beneficial to study reports that have been prepared on historic storms and floods in a region. Such reports can provide information on past storms and the consequences that they may have had on drainage structures.

2.1.4 Intensity-Duration-Frequency Curves

Three rainfall characteristics are important and interact with each other in many hydrologic design problems. Rainfall intensity, duration, and frequency were defined and discussed in the previous section. For use in design, the three characteristics are combined, usually graphically into the intensity-duration-frequency (IDF) curve. Rainfall intensity is graphed as the ordinate and duration as the abscissa. One curve of intensity versus duration is given for each exceedence frequency. IDF curves are location dependent. For example, the IDF curve for Baltimore, MD, is not the same as that for Washington, D.C. The differences, while slight, reflect differences in rainfall characteristics at the two locations. Because of this location dependency, a local IDF curve must be used for hydrologic design work. The development of IDF curves is discussed in Appendix A of HEC-12, Drainage of Highway Pavements (Johnson and Chang, 1984).

IDF curves are plotted on log-log paper and have a characteristic shape. Typically, the IDF curve for a specific exceedence frequency is characteristically curved for small durations, usually 2 hours and shorter, and straight for the longer durations. Thus, the following model can be used to represent the IDF curve for any exceedence frequency:

$$i = \begin{cases} \dfrac{a}{D+b} & \text{for } D \le 2 \text{ h} \\ cD^{d} & \text{for } D > 2 \text{ h} \end{cases} \tag{2.1}$$

where,

i = rainfall intensity, mm/h (in/h)
D = rainfall duration, h
a, b, c, and d = regression constants.

For D less than 2 hours, a linear least-squares relationship is obtained by taking the reciprocal of the equation, which yields:

$$\frac{1}{i} = \frac{D + b}{a} = \frac{1}{a}D + \frac{b}{a} = fD + g$$

Letting y = 1/i, the values of f and g can be fitted using least-squares regression of y on D. The values of a and b are then obtained by algebraic transformation: a = 1/f and b = g/f. For durations longer than 2 hours, the power-model equation is placed in linear form by taking logarithms:

$$\log i = \log c + d \log D$$

$$y = h + dx$$

in which y = log i, h = log c, and x = log D. Once h and d are fitted with least-squares, the value of c is computed by $c = 10^h$.

Volume-duration-frequency (VDF) curves are sometimes provided in hydrologic design manuals. The VDF curve is similar to the IDF curve except the depth of rainfall is graphed as the ordinate. The IDF curve is preferred because many design methods use rainfall intensities rather than rainfall depths.

2.2 HYDROLOGIC ABSTRACTIONS

The collective term given to the various processes that act to remove water from the incoming precipitation before it leaves the watershed as runoff is abstractions. These processes are evaporation, transpiration, interception, infiltration, depression storage, and detention storage. The most important abstractions in determining the surface runoff from a given precipitation event are infiltration, depression storage, and detention storage.

2.2.1 Evaporation

Evaporation is the process by which water from the land and water surfaces is converted into water vapor and returned to the atmosphere. It 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 watershed.

2.2.3 Interception

Interception is the removal of water that 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 2 mm (0.08 in) during a single rainfall event, but usually is nearer 0.5 mm (0.02 in). The quantity of water removed through interception is usually not significant for an isolated storm, but, when added over a period of time, it can be 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

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, no generally acceptable model has been developed to accurately predict infiltration rates or total infiltration volumes for a given watershed.

2.2.5 Depression Storage

Depression storage is the term applied to water that is lost because it becomes trapped in the numerous small depressions that are characteristic of any natural surface. When water temporarily accumulates in a low point with no possibility for escape as runoff, the accumulation is referred to as depression storage. The amount of water that 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 1 to 8 mm (0.04 to 0.3 in) with some values as high as 15 mm (0.6 in) per event. As with evaporation and transpiration, depression storage is generally not directly calculated in highway design.

2.2.6 Detention Storage

Detention storage is water that is temporarily stored in the depth of water necessary for overland flow to occur. The volume of water in motion over the land constitutes the detention storage. The amount of water that 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 2 to 10 mm (0.08 to 0.4 in), but values as high as 50 mm (2 in) have been reported.

2.2.7 Total Abstraction Methods

While the volumes of the individual abstractions may be small, their sum can be hydrologically significant. Therefore, hydrologic methods commonly lump all abstractions together and compute a single value. The SCS curve number method lumps all abstractions together, with the volume equal to the difference between the volumes of rainfall and runoff. The phi-index method assumes a constant rate of abstraction over the duration of the storm. These total abstraction methods simplify the calculation of storm runoff rates.

2.3 CHARACTERISTICS OF RUNOFF

Water that has not been abstracted from the incoming precipitation leaves the watershed as surface runoff. While runoff occurs in several stages, the flow that becomes channelized is the main consideration to highway stream crossing design since it influences 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 characteristics of runoff that are important to drainage design are: (1) the peak discharge or peak rate of flow; (2) the discharge variation with time (hydrograph); (3) the stage-discharge relationship; (4) the total volume of runoff; and (5) the frequency with which discharges of specified magnitudes are likely to be equaled or exceeded (probability of exceedence).

2.3.1 Peak Discharge

The peak discharge, often called peak flow, is the maximum rate of runoff 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 that 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 that 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 the 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 of the storm over 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 2.11 and include the rising and falling limbs, the peak flow, the time to peak, and the time base of the hydrograph. There are several types of hydrographs, such as flow per unit area and stage hydrographs, but all display the same typical variation through time.

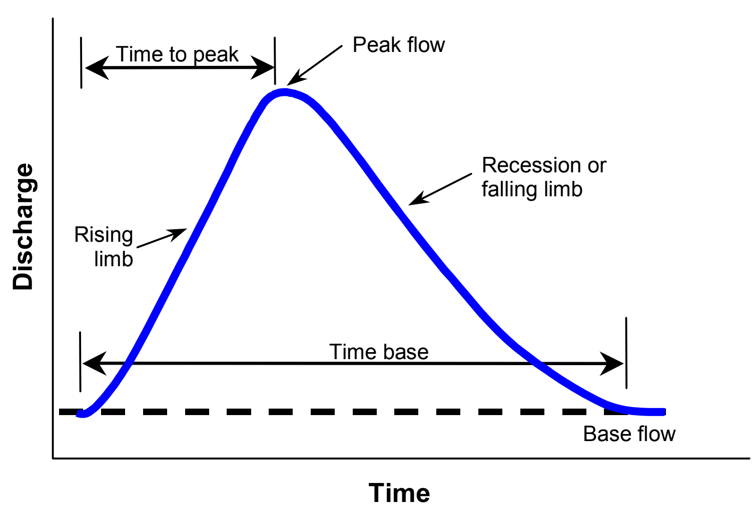

Figure 2.11. Elements of a flood hydrograph

2.3.3 Stage-Discharge

The stage of a river is the elevation of the water surface above some arbitrary datum. The datum can be mean sea level, but can also be set slightly below the point of zero flow in the given stream. The stage of a river is directly related to the discharge, which is the quantity of water passing a given point (see Figure 2.12). As the discharge increases, the stage rises and as the discharge decreases, the stage falls. Generally, discharge is related to stage at a particular point by using a variety of techniques and instrumentation to obtain field measurements of these (and related) parameters.

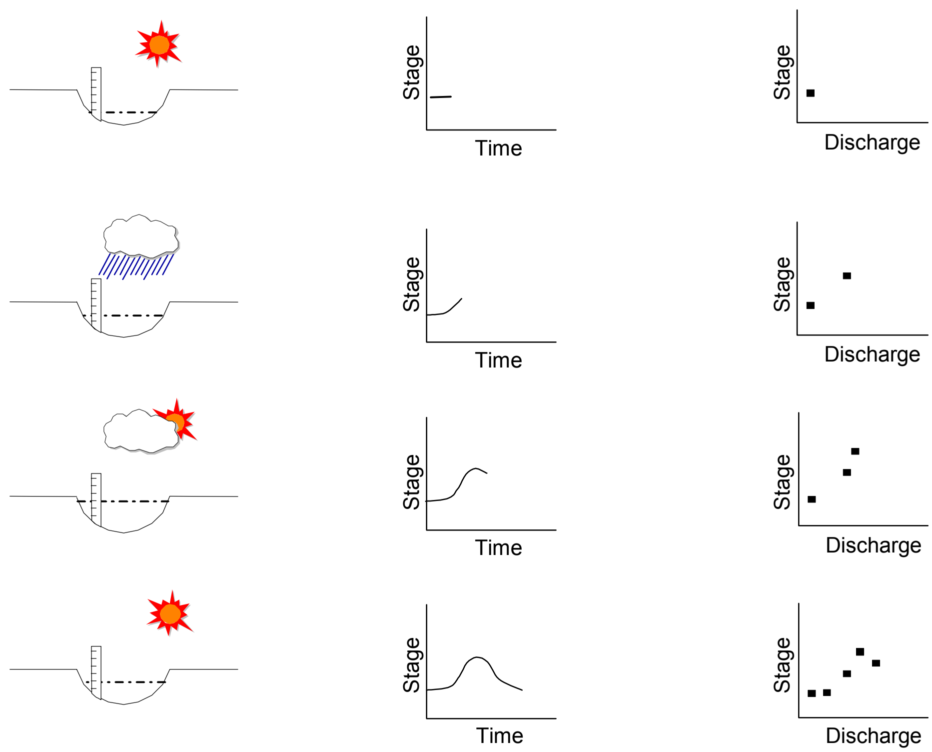

Figure 2.12. Relation between stage and discharge

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 drainage crossing structures. However, flood volume is used in various analyses for other design parameters. Flood volume is most easily determined as the area under the flood hydrograph (Figure 2.11) and is commonly measured in units of cubic meters. The equivalent depth of net rain over the watershed is determined by dividing the volume of runoff by the watershed area.

2.3.5 Frequency

The exceedence frequency is the relative number of times a flood of a given magnitude can be expected to occur on the average over a long period of time. It is usually expressed as a ratio or a percentage. By its definition, frequency is a probabilistic concept and is the probability that a flood of a given magnitude may be equaled or exceeded in a specified period of time, usually 1 year. Exceedence frequency is an important design parameter in that it identifies the level of risk during a specified time interval acceptable for the design of a highway structure.

2.3.6 Return Period

Return period is a term commonly used in hydrology. It is the average time interval between the occurrence of storms or floods of a given magnitude. The exceedence probability (p) and return period (T) are related by:

$$T = \frac{1}{p} \tag{2.2}$$

For example, a flood with an exceedence probability of 0.01 in any one year is referred to as the 100-year flood. The use of the term return period is sometimes discouraged because some people interpret it to mean that there will be exactly T years between occurrences of the event. Two 100-year floods can occur in successive years or they may occur 500 years apart. The return period is only the long-term average number of years between occurrences.

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 gully 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.

A few of the many factors that determine the hydraulic character of the natural drainage system are drainage area, slope, hydraulic roughness, natural and channel storage, drainage density, channel length, antecedent moisture conditions, urbanization, and other factors. 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 that affect the hydraulic character of a given drainage system.

2.4.1 Drainage Area

Drainage area is the most important watershed characteristic that affects runoff. The larger the contributing drainage area, the larger will be the flood runoff (see Figure 2.13a). Regardless of the method utilized to evaluate flood flows, peak flow is directly related to the drainage area.

2.4.2 Slope

Steep slopes tend to result in rapid runoff responses to local rainfall excess and consequently higher peak discharges (see Figure 2.13b). 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 will not be removed as rapidly. The process of infiltration will have more time to affect the rainfall excess, thereby increasing the abstractions and resulting in a reduction of the total volume of rainfall that appears directly as runoff.

Slope is very important in how quickly a drainage channel will convey water and, therefore, it influences 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 the peak flow occurring shortly after the onset of precipitation. If these convective storms occur with a given frequency, 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 that influence the depth and speed of water flowing 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 usually 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 (see Figure 2.13c).

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). A higher roughness results in a higher stage for a given discharge.

The total volume of runoff is virtually independent of hydraulic roughness. An indirect relationship does exist in that higher roughness slows the watershed response and allows some of the abstraction processes more time to affect 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 manmade storage that greatly affects the response to a given precipitation event. Common features that contribute to storage within a watershed are lakes, marshes, heavily vegetated overbank areas, natural or manmade constrictions in the drainage channel that 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 2.13d, 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 effect of storage reservoirs is detailed in Section 7.2.

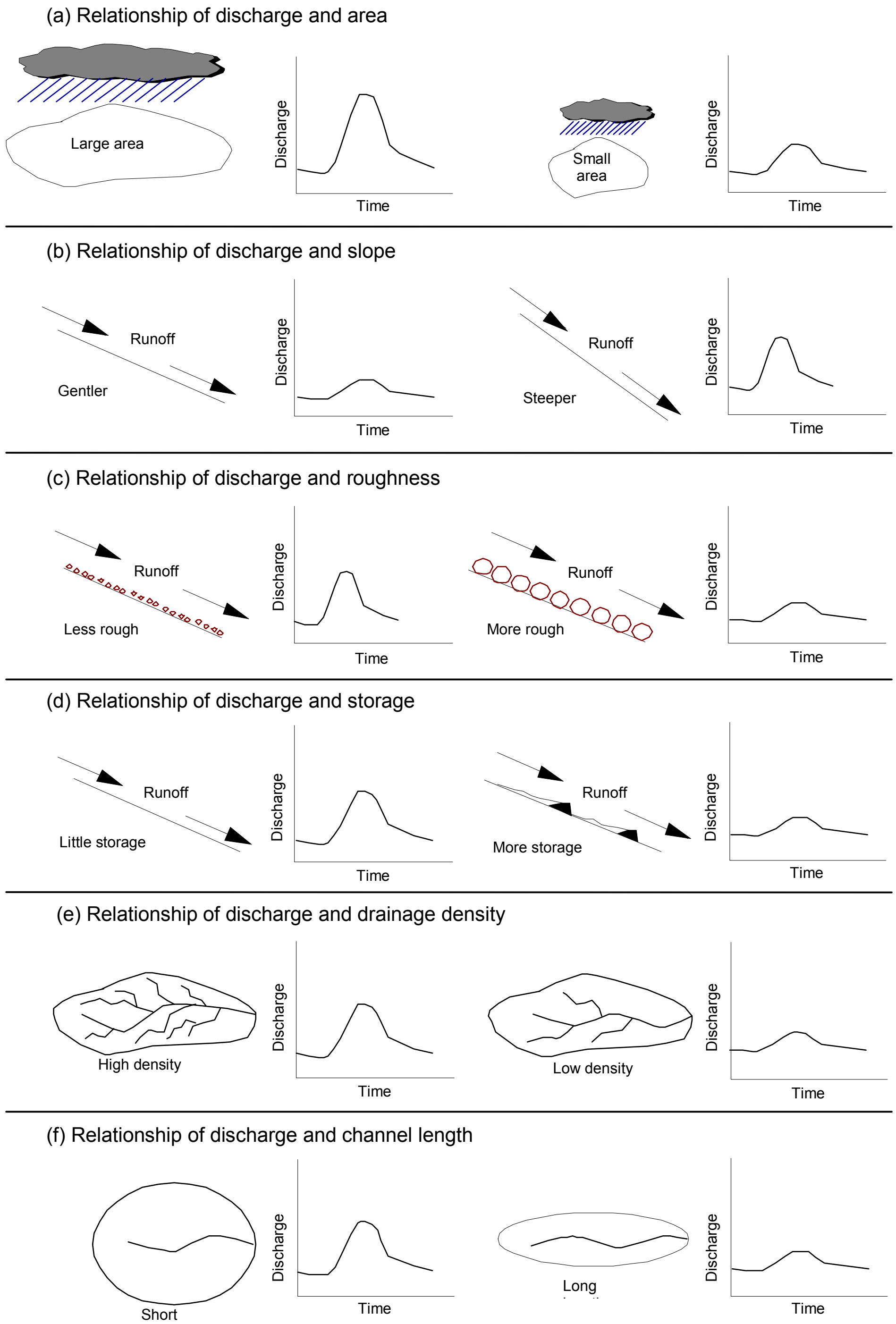

Figure 2-13. Effects of basin characteristics on the flood hydrograph

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 runoff 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 decrease the runoff (as was the case with slope and roughness).

Changes in storage have a definite effect upon the frequency of discharges of given magnitudes. Storage 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. Drainage density is usually assumed to equal the total length of continuously flowing streams divided by the drainage area. It is determined by the topography and the geography of the watershed.

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 overland flow time was relatively long. The mean velocity of runoff is normally lower for overland flow than it is for flow in a well-defined natural channel. High drainage densities are associated with increased response of a watershed leading to higher peak discharges and shorter hydrographs for a given precipitation event (see Figure 2.13e).

Drainage density has a 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 runoff from a given precipitation event.

Changes in drainage density such as with channel improvements in urbanizing watersheds can have an 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 of the response. The higher the drainage density, the more closely related the resultant runoff frequency would be to that of the corresponding precipitation event.

2.4.6 Channel Length

Channel length is an important watershed characteristic. The longer the channel, the more time it takes for water to be conveyed from the headwaters of the watershed to the outlet. Consequently, if all other factors are the same, a watershed with a longer channel length will usually 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 2.13f, 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 floodplain 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 floodplain, 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 moisture levels. The smaller the moisture in the ground at the beginning of precipitation, the lower will be the runoff. Conversely, the larger the moisture content of the soil, the higher the runoff attributable to a particular storm.

2.4.8 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: (I) 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 rates, and travel time. Consequently, peak discharges increase, with the time base of hydrographs becoming shorter and the rising limb rising more quickly.

2.4.9 Other Factors

There can be other factors within the watershed that determine the characteristics of runoff, including the extent and type of vegetation, the presence of channel modifications, and flood control structures. These factors modify 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 ILLUSTRATION OF THE RUNOFF PROCESS

In Section 2.2, several key hydrologic 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 this section. Figures 2.14a through 2.14f show the development of the flood hydrograph from a typical rainfall event.

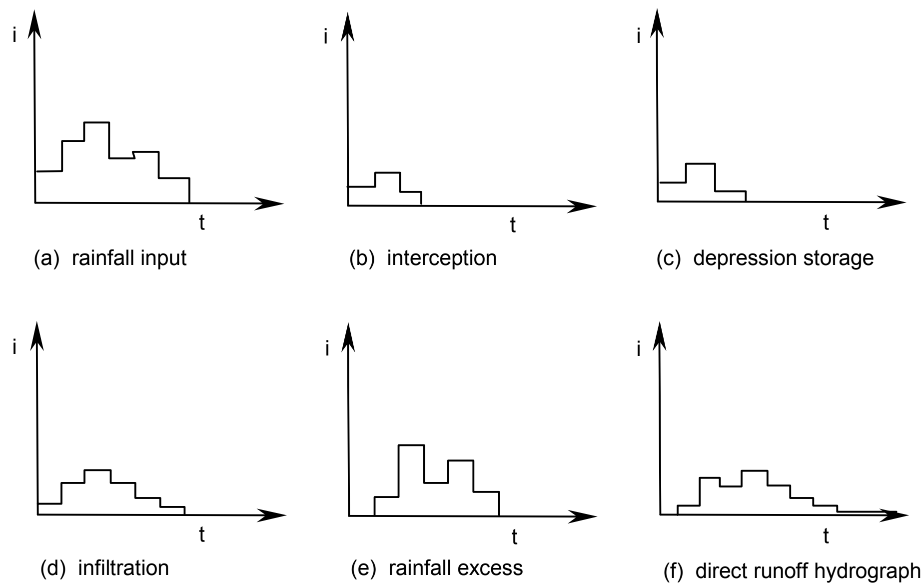

Figure 2.14. The runoff process

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 2.14a. 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 be deleted.

2.5.2 Interception

Figure 2.14b 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 in a relatively short period of time and, once the initial wetting is complete, the interception losses quickly decrease to a lower, nearly constant value. The rainfall that has not been intercepted falls to the ground surface to continue in the runoff process.

2.5.3 Depression Storage

Figure 2.14c illustrates the relative magnitude of depression storage with time. Only the water that is in excess of that necessary to supply the interception is available for depression storage. This is the reason that the depression-storage curve begins at zero. The amount of water that 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 that 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 2.14c. The large depressions take longer to fill and the curve gradually approaches zero when all of the depression storage has been filled. When the rainfall input equals the interception, infiltration, and depression storage, there is no surface runoff.

2.5.4 Infiltration

Infiltration is a complex process, and the rate of infiltration at any point in time depends on many factors. The important point to be illustrated in Figure 2.14d 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 because the upper layers of the soil are drained at a rate that is independent of the rainfall intensity. Most deterministic models, including the phi-index method for estimating infiltration discussed in Section 6.1.4.3, 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 run off from the land surface. As previously defined, this is the rainfall excess and is illustrated in Figure 2.14e. Note how this rainfall excess differs with the actual rainfall input, Figure 2.14a.

The concept of excess rainfall is very important in hydrologic analyses. It is the amount of water available to run off after the initial abstractions and other losses have been satisfied. Except for the losses that may occur during overland and channelized flow, it determines the volume of water that flows past the outlet of a drainage basin. When multiplied by the drainage area, it should be very nearly equal to the volume under the direct runoff hydrograph. The rainfall excess has a direct effect on the outflow hydrograph. It influences the magnitude of the peak flow, the duration of the flood hydrograph, and the shape of the hydrograph.

2.5.6 Detention Storage

A volume of water is detained in temporary (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 is actually the residual of the rainfall input after all abstractions have been satisfied. It is similar in shape to the excess rainfall (see Figure 2.14e), but is extended in time as the detention storage acts on the local runoff.

2.5.8 Outflow Hydrograph

Figure 2.14f illustrates the final outflow hydrograph from the watershed due to the local runoff hydrograph. This final hydrograph is the cumulative effect of all the modifying factors that act on the water as it flows through drainage channels as discussed in Section 2.4. The total volume of water contained under the direct runoff hydrograph of Figure 2.14f and the rainfall excess of

Figure 2.14e are the same, although the position of the outflow hydrograph in time is modified due to the smoothing of the surface runoff and the channel processes.

The processes that have been discussed in the previous sections all act simultaneously to transform the incoming rainfall from that shown in Figure 2.14a to the corresponding outflow hydrograph of Figure 2.14f. 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, areal and time variations and other factors add a new level of complexity to the problem.

2.6 TRAVEL TIME

The travel time of runoff is very important in hydrologic design. In the design of inlets and pipe drainage systems, travel times of surface runoff must be estimated. Some peak discharge methods (Chapter 5) use the time of concentration as input to obtain rainfall intensities from the intensity-duration-frequency curves. Hydrograph times-to-peak, which are in some cases computed from times of concentration, are used with hydrograph methods (Chapter 6). Channel routing methods (Chapter 7) use computed travel times in routing hydrographs through channel reaches. Thus, estimating travel times are central to a variety of hydrologic design problems.

2.6.1 Time of Concentration

The time of concentration, which is denoted as t_c, is defined as the time required for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point. Factors that affect the time of concentration are the length of flow, the slope of the flow path, and the roughness of the flow path. For flow at the upper reaches of a watershed, rainfall characteristics, most notably the intensity, may also influence the velocity of the runoff.

Various methods can be used to estimate the time of concentration of a watershed. When selecting a method to use in design, it is important to select a method that is appropriate for the flow path. Some estimation methods were designed and can be classified as "lumped" in that they were designed and calibrated to be used for an entire watershed; the SCS lag formula is an example of this method. These methods have t_c as the dependent variable. Other methods are intended for one segment of the principal flow path and produce a flow velocity that can be used with the length of that segment of the flow path to compute the travel time on that segment. With this method, the time of concentration equals the sum of the travel times on each segment of the principal flow path.

In classifying these methods so that the proper method can be selected, it is useful to describe the segments of flow paths. Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually 20 to 30 mm (0.8 to 1.2 in) or less. Concentrated flow is runoff that occurs in rills and swales and has depths on the order of 40 to 100 mm (1.6 to 3.9 in). Part of the principal flow path may include pipes or small streams. The travel time through these segments would be computed separately. Velocities in open channels are usually determined assuming bank-full depths.

2.6.2 Velocity Method

The velocity method (sometimes referred to as the segment method) can be used to estimate travel times for sheet flow, shallow concentrated flow, pipe flow, or channel flow. It is based on estimating the travel time from the length and velocity:

$$T_t = \frac{L}{60V} \tag{2.3}$$

where,

T_t = travel time, min
L = flow length, m (ft)
V = flow velocity, m/s (ft/s).

The travel time is computed for the principal flow path. When the principal flow path consists of segments that have different slopes or land covers, the principal flow path should be divided into segments and Equation 2.3 used for each flow segment. The time of concentration is then the sum of travel times:

$$t_c = \sum_{i=1}^{k} T_{t_i} = \sum_{i=1}^{k} \left(\frac{L_i}{60 V_i} \right) \tag{2.4}$$

where,

k = number of segments
i = subscript referring to each flow segment.

Velocity is a function of the type of flow (overland, sheet, rill and gully flow, channel flow, pipe flow), the roughness of the flow path, and the slope of the flow path. Some methods also include a rainfall index such as the 2-year, 24-hour rainfall depth. A number of methods have been developed for estimating the velocity.

2.6.2.1 Sheet-Flow Travel Time

Sheet flow is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. Typically flow depths will not exceed 50 mm (2 in). Such flow occurs over relatively short distances, rarely more than about 90 m (300 ft), but most likely less than 25 m (80 ft). Sheet flow rates are commonly estimated using a version of the kinematic wave equation. The original form of the kinematic wave time of concentration is:

$$t_c = \frac{\alpha}{i^{0.4}} \left(\frac{nL}{\sqrt{S}} \right)^{0.6} \tag{2.5}$$

where,

t_c = time of concentration, min
n = roughness coefficient (see Table 2.1)
L = flow length, m (ft)
i = rainfall intensity, mm/h (in/h), for a storm that has a return period T and duration of t_c minutes
S = slope of the surface, m/m (ft/ft)
α = unit conversion constant equal to 6.9 in SI units and 0.93 in CU units.

Some hydrologic design methods, such as the rational equation, assume that the storm duration equals the time of concentration. Thus, the time of concentration is entered into the IDF curve to find the design intensity. However, for Equation 2.5, i depends on t_c and t_c is not initially known. Therefore, the computation of t_c is an iterative process. An initial estimate of t_c is assumed and used to obtain i from the intensity-duration-frequency curve for the locality. The t_c is computed from Equation 2.5 and used to check the initial value of i. If they are not the same, the process is repeated until two successive t_c estimates are the same.

Table 2.1. Manning's Roughness Coefficient (n) for Overland and Sheet Flow

(SCS, 1986; McCuen, 1989)

n	**Surface Description**
0.011	Smooth asphalt
0.012	Smooth concrete
0.013	Concrete lining
0.014	Good wood
0.014	Brick with cement mortar
0.015	Vitrified clay
0.015	Cast iron
0.024	Corrugated metal pipe
0.024	Cement rubble surface
0.050	Fallow (no residue)
	Cultivated soils
0.060	Residue cover ≤ 20%
0.170	Residue cover > 20%
0.130	Range (natural)
	Grass
0.150	Short grass prairie
0.240	Dense grasses
0.410	Bermuda grass
	Woods*
0.400	Light underbrush
0.800	Dense underbrush

*When selecting n for woody underbrush, consider cover to a height of about 30 mm (0.1 ft). This is the only part of the plant cover that will obstruct sheet flow.

To avoid the necessity to solve for t_c iteratively, the SCS TR-55 (1986) uses the following variation of the kinematic wave equation:

$$t_c = \frac{\alpha}{P_2^{0.5}} \left(\frac{nL}{\sqrt{S}} \right)^{0.8} \tag{2.6}$$

where,

P_2 = 2-year, 24-hour rainfall depth, mm (in)

α = unit conversion constant equal to 5.5 in SI units and 0.42 in CU units.

The other variables are as previously defined. Equation 2.6 is based on an assumed IDF relationship. SCS TR-55 (1986) recommends an upper limit of L = 90 m (300 ft) for using this equation.

2.6.2.2 Shallow Concentrated Flow

After short distances, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using an empirical relationship between the velocity and the slope:

$$V = \alpha k S^{0.5} \tag{2.7}$$

where,

V = velocity, m/s (ft/s)
S = slope, m/m (ft/ft)
k = dimensionless function of land cover (see Table 2.2)
α = unit conversion constant equal to 10 in SI and 33 in CU units.

Table 2.2. Intercept Coefficients for Velocity vs. Slope Relationship (McCuen, 1989)

k	Land Cover/Flow Regime
0.076	Forest with heavy ground litter; hay meadow (overland flow)
0.152	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)
0.213	Short grass pasture (overland flow)
0.274	Cultivated straight row (overland flow)
0.305	Nearly bare and untilled (overland flow); alluvial fans in western mountain regions
0.457	Grassed waterway (shallow concentrated flow)
0.491	Unpaved (shallow concentrated flow)
0.619	Paved area (shallow concentrated flow); small upland gullies

2.6.2.3 Pipe and Channel Flow

Flow in gullies empties into channels or pipes. In many cases, the transition between shallow concentrated flow and open channels may be assumed to occur where either the blue-line stream is depicted on USGS quadrangle sheets (scale equals 1:24000) or when the channel is visible on aerial photographs. Channel lengths may be measured directly from the map or scale photograph. However, depending on the scale of the map and the sinuosity of the channel, a map-derived channel length may be an underestimate. Pipe lengths should be taken from as-built drawings for existing systems and design plans for future systems.

Cross-section information (i.e., depth-area and roughness) can be obtained for any channel reach in the watershed. Manning's equation can be used to estimate average flow velocities in pipes and open channels:

$$V = \frac{\alpha}{n} R^{2/3} S^{1/2} \tag{2.8}$$

where,

V = velocity, m/s (ft/s)
n = Manning's roughness coefficient
R = hydraulic radius, m (ft)

S = slope, m/m (ft/ft)
α = unit conversion constant equal to 1.0 in SI units and 1.49 in CU units.

The hydraulic radius equals the cross-sectional area divided by the wetted perimeter. For a circular pipe flowing full, the hydraulic radius equals one-fourth of the diameter: R = D/4. For flow in a wide rectangular channel, the hydraulic radius is approximately equal to the depth of flow (d): R = d.

Example 2.1: Estimating Time of Concentration with the Velocity Method. Two watershed conditions are indicated, pre- and post-development, and summarized in Table 2.3. In the pre-development condition, the 1.62-hectare (4-acre) drainage area is primarily forested, with a natural channel having a good stand of high grass. In the post-development condition, the channel has been eliminated and replaced with a 380-mm (15-inch) diameter pipe. The solution using SI follows; the process is identical in CU units, but is not included here because the example is straightforward.

For the existing condition, the velocities of flow for the overland and grassed waterway segments can be obtained with Equation 2.7 and Table 2.2. For the slopes given in Table 2.3, the velocities for the first two segments are:

$$V_1 = \alpha k S^{0.5} = 10\,(0.076)(0.01)^{0.5} = 0.076 \ m/s$$

$$V_2 = \alpha k S^{0.5} = 10\,(0.457)(0.008)^{0.5} = 0.409 \ m/s$$

$$V_3 = \frac{1.0}{0.15}\,(0.3)^{0.67}(0.008)^{0.5} = 0.270 \ m/s$$

For the roadside channel, the velocity can be estimated using Manning's equation; a value for Manning's n of 0.15 is obtained from Table 2.1 and a hydraulic radius of 0.3 m is estimated using conditions at the site:

Table 2.3. Characteristics of Principal Flow Path for Example 2.1

Watershed Condition	Flow Segment	Length (m)	Slope (m/m)	Type of Flow
Existing	1	43	0.010	Overland (forest)
	2	79	0.008	Grassed waterway
	3	146	0.008	Roadside channel (high grass, good stand)
Developed	1	15	0.010	Overland (short grass)
	2	15	0.010	Paved
	3	91	0.008	Grassed waterway
	4	128	0.009	Pipe-concrete (D = 380 mm)

Thus the time of concentration can be computed with Equation 2.4:

$$t_c = \frac{43}{0.076} + \frac{79}{0.409} + \frac{146}{0.27} = 566 + 193 + 541 = 1300 \ s \approx 22\,min$$

For the post-development conditions, the flow velocities for the first three segments can be determined with Equation 2.7. For the slopes given in Table 2.3, the velocities are:

$$V_1 = 10(0.213)(0.01)^{0.5} = 0.213 \ m/s$$
$$V_2 = 10(0.619)(0.01)^{0.5} = 0.619 \ m/s$$
$$V_3 = 10(0.457)(0.008)^{0.5} = 0.409 \ m/s$$

Assuming Manning's coefficient equals 0.011 for the concrete pipe and R = D/4, the velocity is:

$$V = \frac{1.0}{0.011}\left(\frac{0.38}{4}\right)^{0.67}(0.009)^{0.5} = 1.8 \ m/s$$

A slope of 0.009 m/m is used since the meandering roadside channel was replaced with a pipe, which resulted in a shorter length of travel and, therefore, a steeper slope. Thus the time of concentration is:

$$t_c = \frac{15}{0.213} + \frac{15}{0.619} + \frac{91}{0.409} + \frac{128}{1.8}$$

$$= 70 + 24 + 222 + 71 = 387 \ s \approx 6 \ min$$

Thus the land development decreased the time of concentration from 22 minutes to 6 minutes.

<u>Example 2.2: Iterative Calculations Using the Kinematic Sheet Flow Equation.</u> Consider the case of overland flow on short grass (n = 0.15) at a slope of 0.005 m/m. Assume the flow length is 50 m. The solution using SI follows; the process is identical in CU units. Equation 2.5 is:

$$t_c = \frac{6.9}{i^{0.4}}\left(\frac{0.15(50)}{\sqrt{0.005}}\right)^{0.6} = \frac{113}{i^{0.4}}$$

The value of i is obtained from an IDF curve for the locality of the project. For this example, the IDF curve of Baltimore is used (see Figure 2.15), and the problem assumes that a 2-year return period is specified. An initial t_c of 12 minutes will be used to obtain the intensity from Figure 2.15. The initial intensity is 116 mm/h. Using the above equation gives a t_c of 17minutes. Since this differs from the assumed t_c of 12 minutes, a second iteration is necessary.

Using a duration of 17 minutes with Figure 2.15 gives a rainfall intensity of 78 mm/h, which, when substituted into the equation, yields an estimated t_c of 20 minutes. Once again, this differs from the assumed value of 17 minutes, so another iteration is required.

For this iteration, the rainfall intensity is found from Figure 2.15 using a duration of 20 minutes. This gives an intensity of 72 mm/h. With the equation, the estimated t_c is again 20 minutes. Therefore, a time of concentration of 20 minutes is used for this flow path.

Example 2.3: Time of Concentration with Iterative Sheet Flow Computations. Figure 2.16a shows the principal flow path for the existing conditions of a small watershed. The characteristics of each section are given in Table 2.4, including the land use/cover, slope, and length. The solution using SI follows; the process is identical in CU units.

The shallow concentrated flow equation is used to compute the velocity of flow for section AB:

$$V = \alpha k S^{0.5} = 10(0.076)(0.07)^{0.5} = 0.2 \ m/s$$

Thus, the travel time is:

$$T_t = \frac{150 \ m}{0.2 \ m/s \ (60)} = 12 \ min$$

For the section BC, Manning's equation is used. For a trapezoidal channel, the hydraulic radius is:

$$R = \frac{A}{P} = \frac{w d + z d^2}{w + 2d\sqrt{1 + z^2}} + \frac{0.3(0.7) + 2(0.7)^2}{0.3 + 2(0.7)\sqrt{1 + (2)^2}} = 0.35 \ m$$

Thus, Manning's equation yields a velocity of:

$$V = \frac{1}{0.040}(0.35)^{0.67}(0.012)^{0.5} = 1.36 \ m/s$$

and the travel time is:

$$T_t = \frac{1050 \ m}{1.36 \ m/s(60)} = 13 \ min$$

Table 2.4. Characteristics of Principal Flow Path for Example 2.3

Watershed Condition	Flow Segment	Length (m)	Slope (m/m)	n	Land Use/Land Cover
Existing	A to B	150	0.07	-	Overland (forest)
	B to C	1050	0.012	0.040	Natural channel (trapezoidal): w = 0.3m, d = 0.7 m, z = 2:1
	C to D	1100	0.006	0.030	Natural channel (trapezoidal): w = 1.25 m, d= 0.7 m, z = 2:1
Developed	E to F	25	0.07	0.013	Sheet flow: i = 47/(0.285 + D) where i[=] mm/h, D[=] h
	F to G	125	0.07	-	Grassed swale
	G to H	275	0.02	-	Paved area
	H to J	600	0.015	0.015	Storm drain (D = 1050 mm)
	J to K	900	0.005	0.019	Open channel (trapezoidal): w = 1.6 m, d = 1 m, z = 1:1

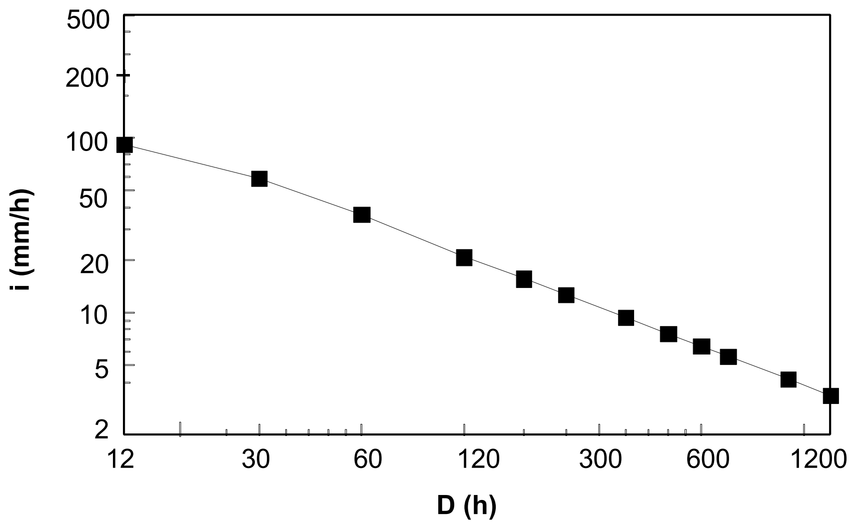

Figure 2.15. Rainfall intensity-duration-frequency curves for selected return periods

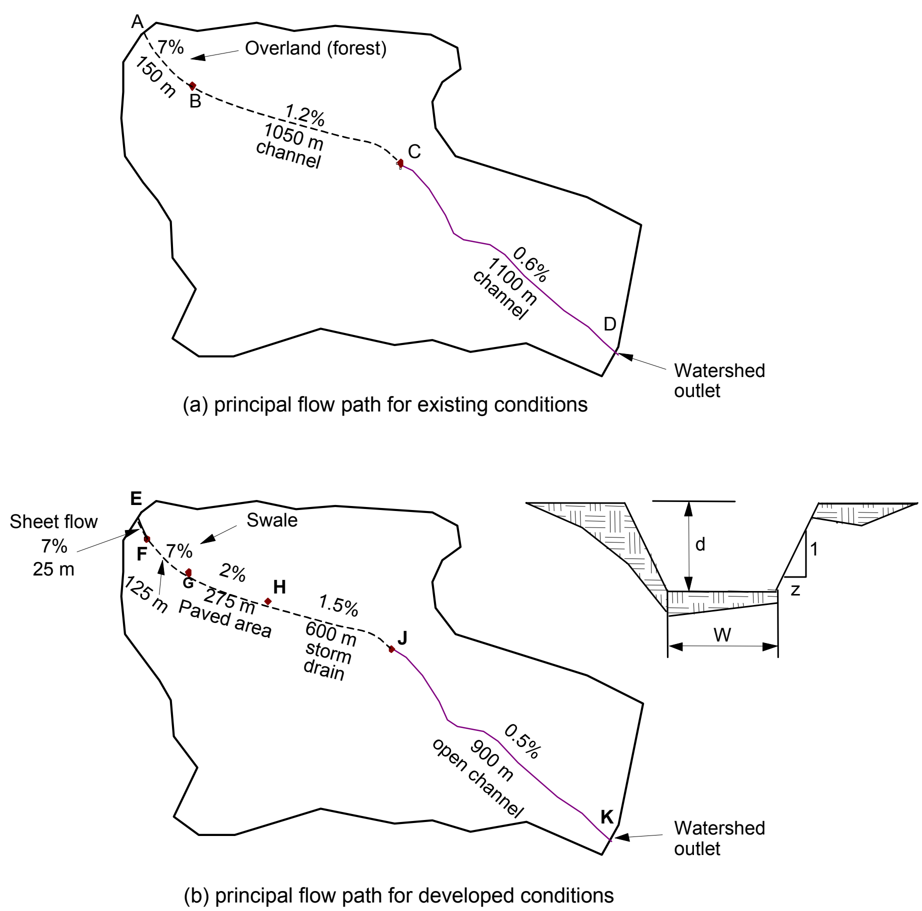

Figure 2.16. Time of concentration estimation

For the section CD, Manning's equation is used. The hydraulic radius is:

$$R = \frac{1.25(0.7) + 2(0.7)^2}{1.25 + 2(0.7)\sqrt{1+(2)^2}} = 0.42 \ m$$

Thus, the velocity is:

$$V = \frac{1}{0.030}(0.42)^{0.67}(0.006)^{0.5} = 1.45 \ m/s$$

and the travel time is:

$$T_t = \frac{1100 \ m}{1.45 \ m/s(60)} = 13 \ min$$

Thus, the total travel time is the sum of the travel times for the individual segments (Equation 2.4):

$$t_c = 12 + 13 + 13 = 38 \ min$$

For the developed conditions, the principal flow path is segmented into five parts (see Figure 2.16b). For the first part of the overland flow portion, the section from E to F, the runoff is sheet flow; thus, the kinematic wave equation (Equation 2.6) is used. Since this is an iterative equation and we will use an intensity associated with the time of concentration for the watershed, we will calculate the travel time for this segment last.

For the section FG, the flow path consists of grass-lined swales. Equation 2.7 can be used to compute the velocity:

$$V = \alpha k S^{0.5} = 10(0.457)(0.07)^{0.5} = 1.21 \ m/s$$

Thus, the travel time is:

$$T_t = \frac{L}{60V} = \frac{125 \ m}{1.21 \ m/s(60)} = 2 \ min$$

For the segment GH, the principal flow path consists of paved gutters. Thus, Equation 2.7 with Table 2.2 is used:

$$V = \alpha k S^{0.5} = 10(0.619)(0.02)^{0.5} = 0.88 \ m/s$$

and the travel time is:

$$T_t = \frac{L}{60 \ V} = \frac{275 \ m}{0.88 \ m/s(60)} = 5 \ min$$

The segment HJ is a 1050-mm (nominally 42-inch) pipe. Thus, Manning's equation is used. The hydraulic radius is one-fourth the diameter (D/4), so the velocity for full flow is:

$$V = \frac{1}{0.015}(0.2625)^{0.67}(0.015)^{0.5} = 3.35 \ m/s$$

and the travel time is:

$$T_t = \frac{L}{60V} = \frac{600 \ m}{3.35 \ m/s(60)} = 3 \ min$$

The final section JK is an improved trapezoidal channel. The hydraulic radius is:

$$R = \frac{w\,d + z\,d^2}{w + 2\,d\,\sqrt{1 + z^2}} = \frac{1.6\,(1) + 1\,(1)^2}{1.6 + 2\,(1)\,\sqrt{1 + 1^2}} = 0.59 \ m$$

Manning's equation is used to compute the velocity:

$$V = \frac{1}{0.019}(0.59)^{0.67}(0.005)^{0.5} = 2.61 \ m/s$$

and the travel time is:

$$T_t = \frac{L}{60\,V} = \frac{900 \ m}{2.61 \ m/s\,(60)} = 6 \ min$$

Thus, the total travel time through the four segments (excluding the first segment) is:

$$t_c = \sum T_t = 2 + 5 + 3 + 6 = 16 \ min$$

Therefore, we know that the time of concentration will be 16 min plus the time of travel over the sheet flow segment EF. For short durations at the location of this example, the 2-year IDF curve is represented by the following relationship between i and D:

$$i = \frac{47}{0.285 + D}$$

where,

i = intensity, mm/h
D = duration, h.

Iteration 1: Assume that travel time on the sheet flow segment is 2 minutes. Therefore, t_c = D = 16 + 2 = 18 min. The 2-year IDF curve is used to estimate the intensity:

$$i = \frac{47}{0.285 + D} = \frac{47}{0.285 + 18/60} = 80 \ mm/h$$

Consequently, Equation 2.6 yields an estimate of the travel time:

$$T_t = \frac{6.9}{80^{0.4}}\left(\frac{0.013(25)}{\sqrt{0.07}}\right)^{0.6} = 1 \ min$$

Since we assumed 2 min for this segment, a second iteration will be performed using the new estimate.

Iteration 2: Assume $t_c = D = 16 + 1 = 17$ min

$$i = \frac{47}{0.285 + 17/60} = 83\ mm/h$$

$$T_t = \frac{6.9}{83^{0.4}} \left(\frac{0.013(25)}{\sqrt{0.07}} \right)^{0.6} = 1\ min$$

The change in rainfall intensity did not change the travel time for this segment (rounded to the nearest minute); therefore, the computations are completed. The time of concentration for the post-developed condition is 17 min. This t_c is 45 percent of the t_c for the existing conditions.

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CHAPTER 3

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 detailed analysis is to be performed to obtain parameters for final design. Once 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 stream flow records. Depending on the size and scope of the project, it may even be necessary to seek out historical data on floods to better define the stream flow record. Occasionally, the collection of raw data may be necessitated by the project purposes.

If data needs are clearly identified, the effort necessary for data 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; however, data needs vary with the method of analysis and 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 are 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 could 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:

- Site investigations and field surveys.
- Published and electronic files of federal agencies such as the National Weather Service (NWS), U.S. Geological Survey (USGS), National Resources Conservation Service (NRCS), among others. (The NRCS was formerly known as the SCS, or Soil Conservation Service.)
- Files of state and local agencies such as state highway departments, water agencies, and various planning organizations.
- Other published or electronically available reports and documents.

The Internet has become a significant source of information for hydrologic data. While the Internet continues to grow and change as a resource, three federal agency sites are of particular note:

- www.usgs.gov for stream discharge and stage data
- www.nws.noaa.gov for precipitation data
- www.nrcs.usda.gov for soils data

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 that 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 Model Drainage Manual and in Federal Highway Administration (FHWA) guidelines.

Typical data that 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 that 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 that 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 procedures:

I. The individual in charge of the drainage aspects of the field site survey should have a general knowledge of drainage design.

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

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

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 only 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 is the records and reports that 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 that 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 that 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.

Federal agencies that collect data include the U.S. Army Corps of Engineers, the USGS, the NRCS, 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 that should be considered by the highway designer. Floods are noteworthy events and, very often, after a flood occurs, specific information such as high-water elevations are recorded. Sources of such information include newspapers, magazines, state historical societies or universities, and publications by any of several federal agencies. Previous storms or flood events of historic proportion have been very thoroughly documented by the USGS, the Corps of Engineers, and the NWS. USGS reports documenting historic floods are summarized by Thomas (1987). Such publications 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 preceding paragraphs may provide hydrologic data in a form suitable for analysis by the highway designer. Other sources of data will provide information of a more basic nature. An example is the data available from the USGS for the network of stream gauging stations that this agency maintains throughout the country. The stream-gaging program operated by the USGS is described by Condes (1992). 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) stream flow records; (2) precipitation records; (3) soil types; (4) land use; and (5) other types of basic data needed for hydrologic analysis.

3.1.2.1 Stream Flow Data

The major source of stream flow information is the USGS, an agency charged with collecting and documenting the data. In 1994, the USGS collected data at 7,292 stream-gauging stations nationwide. Their computer database holds mean daily-discharge data for about 18,500 locations (Wahl, et al., 1995). USGS compiles and publishes this data in both Water Supply Papers and on the USGS web site. The database contains a peak flow data retrieval capability that 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).

Also, the Corps of Engineers and the Bureau of Reclamation collect stream flow 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 governments, utility companies, water-intensive industries, and academic or research institutions.

3.1.2.2 Precipitation Data

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

Each WSFO uses these data and information obtained via satellite and other means to forecast the weather for its area of responsibility. In addition to the WSFOs, the NWS maintains a network of River Forecast Centers (RFCs) that prepare river and flood forecasts for about 2,500 communities. These two organizational units of the National Weather Service are an excellent source of data and information.

The highway engineer can also obtain data from a regional office of 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 National Climatic Data Center (NCDC). The NCDC 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.

3.1.2.3 Soil Type Data

Information on the type of soil that is characteristic of a particular region is often needed as a basic input in hydrologic evaluations. The major source of soil information is the NRCS, which is actively engaged in the classification and mapping of the soils across the U.S. Soil maps have been prepared for most of the counties in the country. The highway designer should contact the NRCS or county extension service to determine the availability of this data.

3.1.2.4 Land-Use Data

Land-use data are 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 USGS, 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 the geographic area of interest, and the types of information that they collect, publish, or record.

3.1.2.5 Miscellaneous Basic Data

Aerial photographs are an excellent source of hydrologic information and the SCS 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 other services and products too numerous to list.

Other types of basic data that 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 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. The water resources centers located at most land grant universities can also assist in data source identification.

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 that the designer will need. This is the broad collection of general information sources that are invaluable aids in hydrologic analyses. Among them are general references such as textbooks, drainage or hydrology manuals of state or federal agencies, hydrologic 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 that could improve the accuracy or efficiency of hydrologic analyses.

3.2 ADEQUACY OF DATA

Once the needed hydrologic data have 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 that 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. 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 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 stream flow and precipitation, need to be evaluated for hydrologic homogeneity and summarized before use. Maps, aerial photographs, Landsat images, and land-use studies should 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 that are used so frequently that standardized formats are appropriate. These will be illustrated with examples in subsequent sections of this document.

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

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

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 are either included in the documentation or adequately referenced so that they may be quickly retrieved. This is true, whether or not the data were used in the subsequent analysis, since they could be very useful in a future study.

It is also important that data analyses be presented in the documentation. If several different methods were used, 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.

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 that 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 the 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.

CHAPTER 4

PEAK FLOW FOR GAGED SITES

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

- Gaged sites: the site is at or near a gaging station, and the stream flow record is fairly complete and of sufficient length to be used to provide estimates of peak discharges.

- Ungaged sites: the site is not near a gaging station or the stream flow record is not adequate for analysis.

Sites that are located at or near a gaging station, but that have incomplete or very short records represent special cases. For these situations, peak discharges for selected frequencies 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.

The USGS Interagency Advisory Committee on Water Data Bulletin 17B (1982) is a guide that "describes the data and procedures for computing flood flow frequency curves where systematic stream gaging records of sufficient length (at least 10 years) to warrant statistical analysis are available as the basis for determination." The guide was intended for use in analyzing records of annual flood peak discharges, including both systematic records and historic data. The document iscommonly referred to simply as “Bulletin 17B”.

Methods for making flood peak estimates can be separated on the basis of the gaged vs. ungaged classification. If gaged data are available at or near the site of interest, the statistical analysis of the gaged data is generally the preferred method of analysis. Where such data are not available, estimates of flood peaks can be made using either regional regression equations or one of the generally available empirical equations. If the assumptions that underlie the regional regression equations are valid for the site of interest, their use is preferred to the use of empirical equations. The USGS has developed and published regional regression equations for estimating the magnitude and frequency of flood discharges for all states and the Commonwealth of Puerto Rico (Jennings, et al., 1994). Empirical approaches include the rational equation and the SCS graphical peak discharge equation.

This chapter 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 the empirical equations applicable to ungaged sites are discussed in Chapter 5.

4.1 RECORD LENGTH REQUIREMENTS

Analysis of gaged data permits an estimate of the peak discharge in terms of its probability or frequency of exceedence 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. Bulletin 17B (1982) suggests that at least 10 years of record are necessary to warrant a statistical analysis by methods presented therein.

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

4.2 STATISTICAL CHARACTER OF FLOODS

The concepts of populations and samples are fundamental to statistical analysis. A population that 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.

An entire branch of statistics 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. Some of the methods available for the analysis of data are discussed below and illustrated with actual peak flow data.

Before analyzing data, it is necessary that they 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: magnitude; time of occurrence; and geographic location.

4.2.1 Analysis of Annual and Partial-Duration Series

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, 29 annual peak discharges for Mono Creek near Vermilion Valley, California, are listed in Table 4.1.

Another method used in flood data arrangement is the partial-duration series. This procedure uses all peak flows above some base value. For example, the partial-duration series may consider all flows above the discharge of approximately bankfull stage. The USGS sets the base for the partial-duration series so that approximately three peak flows, on average, exceed the base each year. Over a 20-year period of record, this may yield 60 or more floods compared to 20 floods in the annual series. The record contains both annual peaks and partial-duration peaks for unregulated watersheds. Figure 4.1 illustrates a portion of the record for Mono Creek containing both the highest annual floods and other large secondary floods.

Table 4.1. Analysis of Annual Flood Series, Mono Creek, CA

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

Location: Latitude 37°22'00", Longitude 118° 59' 20", 1.6 km (1 mi) downstream from lower end of Vermilion Valley and 9.6 km (6.0 mi) downstream from North Fork

Area: 238.3 km^2 (92 mi^2)

Remarks: diversion or regulation

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

Year	Annual Maximum (m^3/s)	Smoothed Series (m^3/s)	Annual Maximum (ft^3/s)	Smoothed Series (ft^3/s)
1922	39.4	-	1,390	-
1923	26.6	-	940	-
1924	13.8	27.8	488	982
1925	30.0	28.0	1,060	988
1926	29.2	28.9	1,030	1,022
1927	40.2	30.4	1,420	1,074
1928	31.4	29.2	1,110	1,031
1929	21.2	26.4	750	931
1930	24.0	26.4	848	931
1931	14.9	27.7	525	979
1932	40.2	25.8	1,420	909
1933	38.2	27.9	1,350	986
1934	11.4	30.9	404	1,093
1935	34.8	29.8	1,230	1,051
1936	30.0	32.1	1,060	1,133
1937	34.3	32.8	1,210	1,160
1938	49.8	32.3	1,760	1,140
1939	15.3	34.3	540	1,212
1940	32.0	34.1	1,130	1,204
1941	40.2	32.3	1,420	1,140
1942	33.1	34.1	1,170	1,203
1943	40.8	35.4	1,440	1,251
1944	24.2	32.5	855	1,149
1945	38.8	31.5	1,370	1,113
1946	25.8	28.1	910	992
1947	28.0	28.4	988	1,004
1948	23.7	26.9	838	950
1949	25.9	-	916	-
1950	31.2	-	1,100	-

Partial-duration series are used primarily in defining annual flood damages when more than one event that causes flood damages can occur in any year. If the base for the partial-duration series conforms approximately to bankfull stage, the peaks above the base are generally flood-damaging events. The partial-duration series avoids a problem with the annual-maximum series, specifically that annual-maximum series analyses ignore floods that are not the highest flood of that year even though they are larger than the highest floods of other years. While partial-duration series produce larger sample sizes than annual maximum series, they require a criterion that defines peak independence. Two large peaks that are several days apart and separated by a period of lower flows may be part of the same hydrometeorological event and, thus, they may not be independent events. Independence of events is a basic assumption that underlies the method of analysis.

If these floods are ordered in the same manner as in an annual series, they can be plotted as illustrated in Figure 4.2. By separating out the peak annual flows, the two series can be compared as also shown in Figure 4.2, where it is seen that, for a given rank (from largest to smallest) order, m, the partial-duration series yields a 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 rank 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:

$$T_B = \frac{1}{\ln T_A \quad \ln(T_A - 1)} \qquad (4.1)$$

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 4.3.

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 4.2).

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, 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.

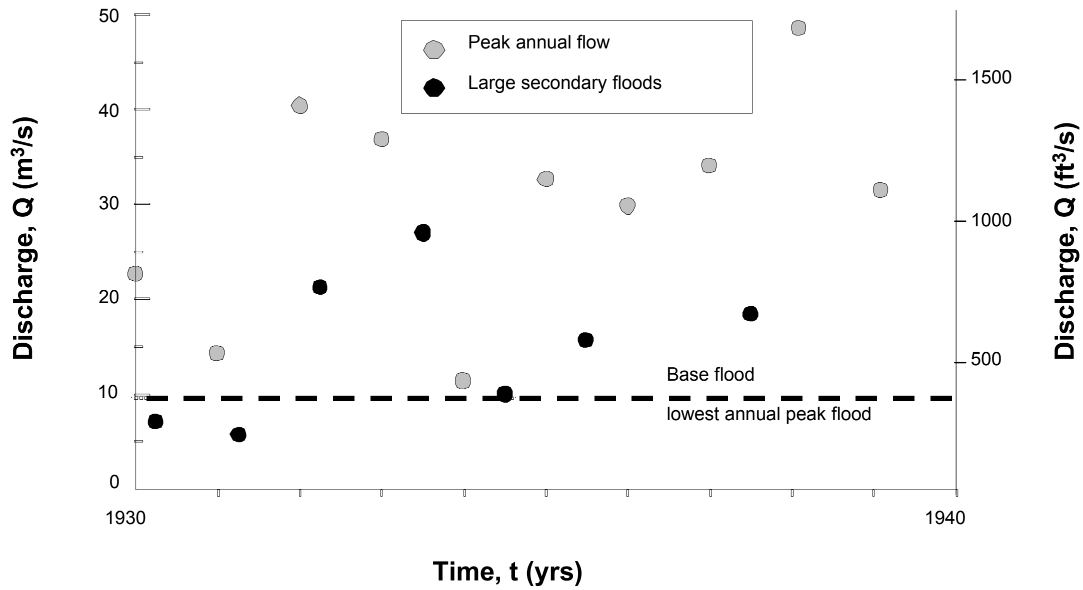


Figure 4.1. Peak annual and other large secondary flows, Mono Creek, CA

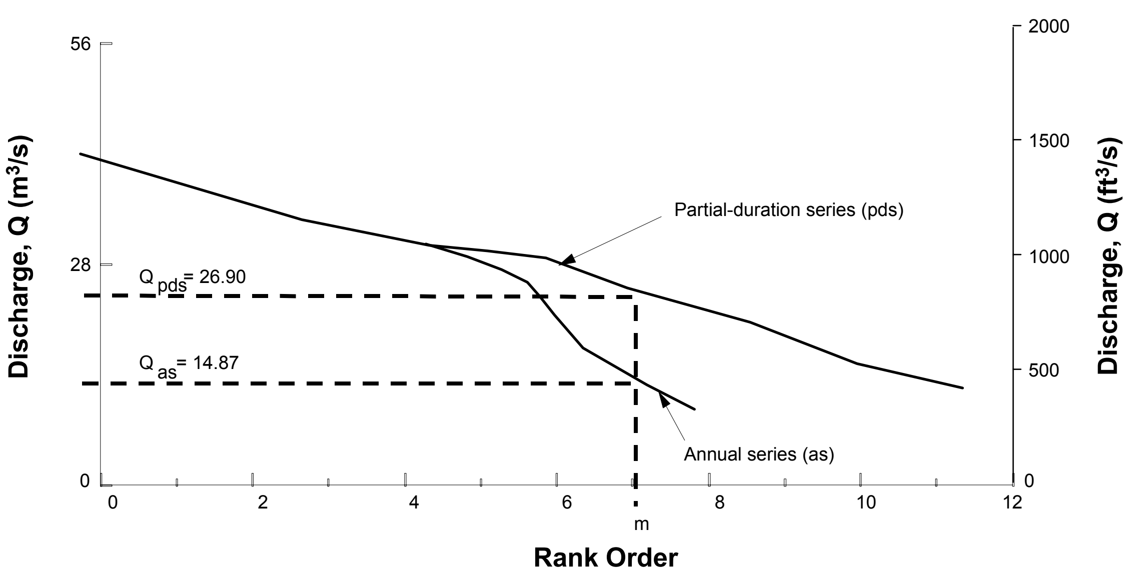


Figure 4.2. Annual and partial-duration series

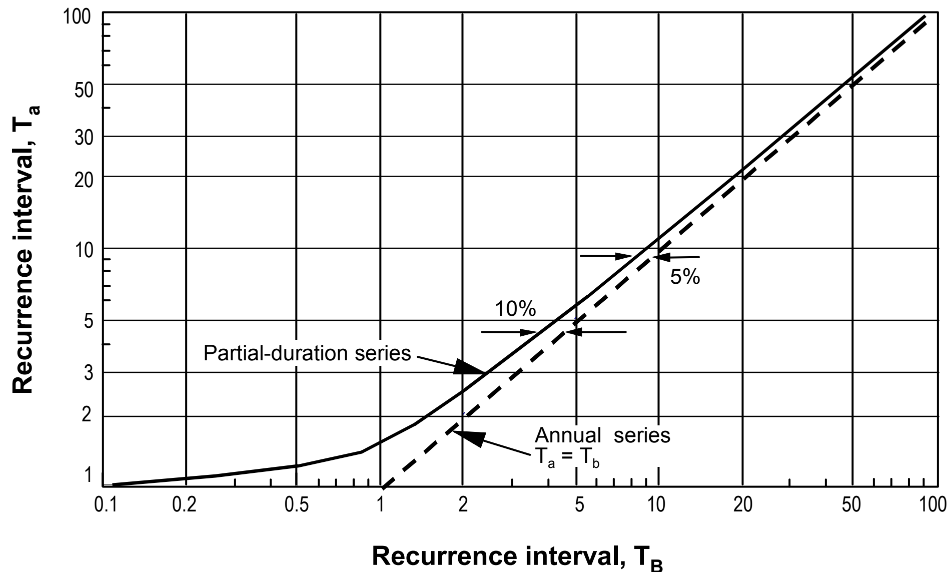


Figure 4.3. Relation between annual and partial-duration series

Table 4.2. Comparison of Annual and Partial-Duration Curves

Number of Years Flow is Exceeded per Hundred Years
(from Beard, 1962)

Annual-event	Partial-duration
1	1.00
2	2.02
5	5.10
10	10.50
20	22.30
30	35.60
40	51.00
50	69.30
60	91.70
63	100.00
70	120.00
80	161.00
90	230.00
95	300.00

4.2.2 Detection of Nonhomogeneity in the Annual Flood Series

Frequency analysis is a method based on order-theory statistics. Basic assumptions that should be evaluated prior to performing the analysis are:

The data are independent and identically distributed random events.

1. The data are from the sample population.
2. The data are assumed to be representative of the population.

3. The process generating these events is stationary with respect to time.

Obviously, using a frequency analysis assumes that no measurement or computational errors were made. When analyzing a set of data, the validity of the four assumptions can be statistically evaluated using tests such as the following:

- Runs test for randomness
- Mann-Whitney U test for homogeneity
- Kendall test for trend
- Spearman rank-order correlation coefficient for trend

The Kendall test is described by Hirsch, et al. (1982). The other tests are described in the British Flood Studies Report (National Environmental Research Council, 1975) and in the documentation for the Canadian flood-frequency program (Pilon and Harvey, 1992). A work group for revising USGS Bulletin 17B (1982) is currently writing a report that documents and illustrates these tests.

Another way to arrange data is according to their 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 4.1 are arranged according to year of occurrence rather than magnitude and plotted in Figure 4.4.

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 that are used in trend analysis. The two most commonly used in hydrologic analysis are the moving-average method and the methods of curve fitting. A major difference between the moving-average method and curve fitting is that the moving-average method does not provide a mathematical equation for making estimates. It only provides a tabular or graphical summary from which a trend can be subjectively assessed. Curve fitting can provide an equation that can be used to make estimates. The various methods of curve fitting are discussed in more detail by Sanders (1980) and McCuen (1993).

The method of moving averages is presented here. Moving-average filtering reduces the effects of random variations. The method is based on the premise that the systematic component of a time series exhibits autocorrelation (i.e., correlation between nearby measurements) while the random fluctuations are not autocorrelated. Therefore, the averaging of adjacent measurements will eliminate the random fluctuations, with the result converging to a qualitative description of any systematic trend that is present in the data.

In general, the moving-average computation uses a weighted average of adjacent observations to produce a new time series that consists of the systematic trend. Given a time series Y_i, the filtered series $\hat{Y}_i$ is derived by:

$$\hat{Y}_i = \sum_{j=1}^{m} w_j Y_{i-k+j-1} \quad \text{for} \quad i = (k+1), (k+2), \ldots, (n-k) \tag{4.2}$$

where,

m = the number of observations used to compute the filtered value (i.e., the smoothing interval)

w_j = the weight applied to value j of the series Y.

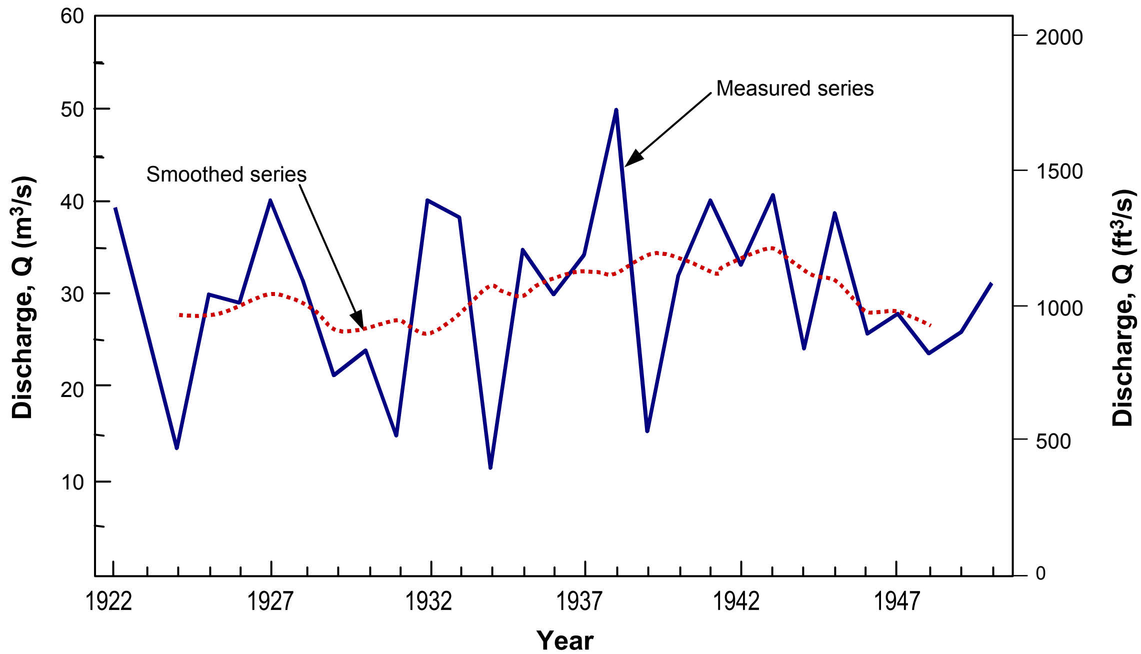


Figure 4.4. Measured and smoothed flood series for Mono Creek, CA

The smoothing interval should be an odd integer, with 0.5 (m-1) values of Y before observation i and 0.5 (m-1) values of Y after observation i is used to estimate the smoothed value $\hat{Y}$. A total of 2*k observations are lost; that is, while the length of the measured time series equals n, the smoothed series, $\hat{Y}$, has (n - 2k) values. The simplest weighting scheme would be the arithmetic mean (i.e., w_j = 1/m). Other weighting schemes give the greatest weight to the central point in the interval, with successively smaller weights given to points farther removed from the central point.

Moving-average filtering has several disadvantages. First, as described above, the approach loses 2*k observations, which may be a very limiting disadvantage for short record lengths. Second, a moving-average filter is not itself a mathematical representation, and thus forecasting with the filter is not possible; a structural form must still be calibrated to forecast any systematic trend identified by the filtering. Third, the choice of the smoothing interval is not always obvious, and it is often necessary to try several values in order to provide the best separation of systematic and random variation. Fourth, if the smoothing interval is not properly selected, it is possible to eliminate some of the systematic variation with the random variation.

A moving-average filter can be used to identify the presence of either a trend or a cycle. The smoothed series will enable the form of the trend or the period of the cycle to be estimated. A model can be developed to represent the systematic component and the model coefficients evaluated with a numerical fitting method.

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 and past events are better understood.

Two examples will be used to demonstrate the use of moving-average smoothing. In both cases, a 5-year smoothing interval was used. Three-year intervals were not sufficient to clearly show the trend, and intervals longer than 5 years did not improve the ability to interpret the results.

Example 4.1. Table 4.1 contains the 29-year annual flood series for Mono Creek, CA; the series is shown in Figure 4.4. The calculated smoothed series is also listed in Table 4.1 and shown in Figure 4.4. The trend in the smoothed series is not hydrologically significant, which suggests that rainfall and watershed conditions have not caused a systematic trend during the period of record.

Example 4.2. Table 4.3 contains the 24-year annual flood series and smoothed series for Pond Creek, KY; the two series are shown in Figure 4.5. The Pond Creek watershed became urbanized in the late 1950s. Thus, the flood peaks tended to increase. This is evident from the obvious trend in the smoothed series during the period of urbanization. It appears that urbanization caused at least a doubling of flood magnitudes. While the smoothing does not provide a model of the effects of urbanization, the series does suggest the character of the effects of urbanization. Other possible causes of the trend should be investigated to provide some assurance that the urban development was the cause.

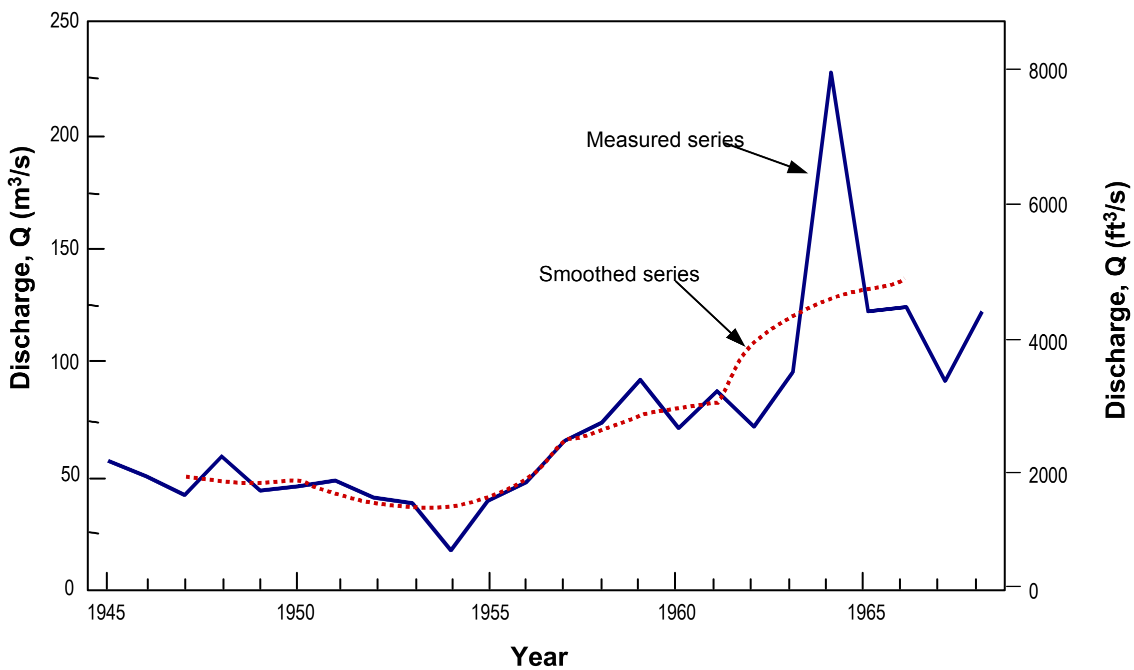


Figure 4.5. Measured and smoothed series for annual peak flows, Pond Creek, KY

Table 4.3. Computation of 5-year Moving Average of Peak Flows, Pond Creek, KY

Year	Annual Maximum (m^3/s)	Smoothed Series (m^3/s)	Annual Maximum (ft^3/s)	Smoothed Series (ft^3/s)
1945	56.7	-	2,002	-
1946	49.3	-	1,741	-
1947	41.4	49.8	1,462	1,760
1948	58.4	47.5	2,062	1,678
1949	43.4	47.2	1,532	1,668
1950	45.1	47.0	1,593	1,660
1951	47.9	42.8	1,691	1,513
1952	40.2	37.6	1,419	1,328
1953	37.7	36.4	1,331	1,286
1954	17.2	36.3	607	1,280
1955	39.1	41.2	1,381	1,454
1956	47.0	48.3	1,660	1,706
1957	64.9	63.4	2,292	2,237
1958	73.4	69.7	2,592	2,460
1959	92.4	77.7	3,263	2,744
1960	70.6	79.0	2,493	2,790
1961	87.3	83.4	3,083	2,944
1962	71.4	110.4	2,521	3,897
1963	95.2	120.7	3,362	4,261
1964	227.3	128.0	8,026	4,520
1965	122.1	132.0	4,311	4,661
1966	124.1	137.4	4,382	4,853
1967	91.3	-	3,224	-
1968	122.4	-	4,322	-

4.2.3 Arrangement by Geographic Location

The primary purpose of arranging flood data by geographic area is to develop a database 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 climatic conditions. Similarity of physiographic features means that basin slopes, shapes, stream density, ground cover, geology, and hydrologic abstractions are similar among watersheds in the same region.

Some of these parameters are described quantitatively in a variety of ways while others are totally subjective. 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 that describe the distribution of floods from different watersheds. These properties, which are described more fully in later parts of this section, include the variance, standard deviation, and coefficient of skew. Other methods 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 the residuals of 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 Probability 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 exceedence of a single peak flow, Q_A, is approximated by the relative number of exceedences of Q_A after a long series of observations, i.e.,

$$P_r(Q_A) = \frac{n_1}{n} = \frac{\text{No. of exceedences of some flood magnitude}}{\text{No. of observations (if large)}} \qquad (4.3)$$

where,

n_1 = the frequency

n_1/n = relative frequency of Q_A.

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 nonexceedence (or failure) of an event such as peak flow, Q_A, is given by:

$$P_r(\text{not } Q_A) = \frac{n - n_1}{n} = 1 - \frac{n_1}{n} = 1 - P_r(Q_A) \qquad (4.4)$$

Combining Equations 4.3 and 4.4 yields:

$$P_r(Q_A) + P_r(\text{not } Q_A) = 1 \qquad (4.5)$$

or the probability of an event being exceeded is between 0 and 1 (i.e., $0 \leq Pr(Q_A) \leq 1$). If an event is certain to occur, it has a probability of 1, and if it cannot occur at all, it has a probability of 0.

Given two independent flows, Q_A and Q_B, the probability of the successive exceedence of both Q_A and Q_B is given by:

$$P_r(Q_A \text{ and } Q_B) = P_r(Q_A)\, P_r(Q_B) \qquad (4.6)$$

If the exceedence of a flow Q_A excludes the exceedence of another flow Q_2, the two events are said to be mutually exclusive. For mutually exclusive events, the probability of exceedence of either Q_A or Q_B is given by:

$$P_r(Q_A \text{ or } Q_B) = P_r(Q_A) + P_r(Q_B) \qquad (4.7)$$

4.2.5 Return Period

If the exceedence probability of a given annual peak flow or its relative frequency determined from Equation 4.3 is 0.2, this means that there is a 20 percent chance that this flood, over a long period of time, will be exceeded in any one year. Stated another way, this flood will be exceeded on an average of once every 5 years. That time interval is called the return period, recurrence interval, or exceedence frequency.

The return period, T_r, is related to the probability of exceedence by:

$$T_r = \frac{1}{P_r(Q_A)} \tag{4.8}$$

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 5 years. As noted, the flood has a 20 percent probability of being exceeded in any year, and there is no preclusion of the 5-year flood being exceeded in several consecutive years. Two 5-year floods can occur in two consecutive years; there is also a probability that a 5-year flood may not be exceeded in a 10-year period. The same is true for any flood of specified return period.

4.2.6 Estimation of Parameters

Flood frequency analysis uses sample information to fit a population, which is a probability distribution. These distributions have parameters that must be estimated in order to make probability statements about the likelihood of future flood magnitudes. A number of methods for estimating the parameters are available. USGS Bulletin 17B (1982) uses the method of moments, which is just one of the parameter-estimation methods. The method of maximum likelihood is a second method.

The method of moments equates the moments of the sample flood record to the moments of the population distribution, which yields equations for estimating the parameters of the population as a function of the sample moments. As an example, if the population is assumed to follow distribution f(x), then the sample mean ($\overline{X}$) could be related to the definition of the population mean (μ):

$$\overline{X} = \int_{-\infty}^{\infty} x f(x) dx \tag{4.9}$$

and the sample variance (S^2) could be related to the definition of the population variance (σ^2):

$$S^2 = \int_{-\infty}^{\infty} (X - \mu)^2 f(x) dx \tag{4.10}$$

Since f(x) is a function that includes the parameters (μ and σ^2), the solution of Equations 4.9 and 4.10 will be expressions that relate $\overline{X}$ and S^2 to the parameters μ and σ^2.

While maximum likelihood estimation (MLE) is not used in USGS Bulletin 17B (1982) and it is more involved than the method of moments, it is instructive to put MLE in perspective. MLE defines a likelihood function that expresses the probability of obtaining the population parameters given that the measured flood record has occurred. For example, if μ and σ are the population parameters and the flood record X contains N events, the likelihood function is:

$$L(\mu,\sigma \mid X_1, X_2, \ldots, X_N) = \prod_{i=1}^{N} f(X_i \mid \mu,\sigma) \tag{4.11}$$

where $f(X_I \mid \mu, \sigma)$ is the probability distribution of X as a function of the parameters. The solution of Equation 4.11 will yield expressions for estimating μ and σ from the flood record X.

4.2.7 Frequency Analysis Concepts

Future floods cannot be predicted with certainty. Therefore, their magnitude and frequency are treated using probability concepts. To do this, a sample of flood magnitudes are obtained and analyzed for the purpose of estimating a population that can be used to represent flooding at that location. The assumed population is then used in making projections of the magnitude and frequency of floods. It is important to recognize that the population is estimated from sample information and that the assumed population, not the sample, is then used for making statements about the likelihood of future flooding. The purpose of this section is to introduce concepts that are important in analyzing sample flood data in order to identify a probability distribution that can represent the occurrence of flooding.

4.2.7.1 Frequency Histograms

Frequency distributions are used to facilitate an analysis of sample data. A frequency distribution, which is sometimes presented as a histogram, is an arrangement of data by classes or categories with associated frequencies of each class. The frequency distribution shows the magnitude of past events for certain ranges of the variable. Sample probabilities can also be computed by dividing the frequencies of each interval by the sample size.

A frequency distribution or histogram 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 5 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 precise rule concerning the number of class intervals to select, but the following guidelines may be helpful:

1. The class intervals should not overlap, and there should be no gaps between the bounds of the intervals.

2. The number of class intervals should be chosen so that most class intervals have at least one event.

3. It is preferable that the class intervals are of equal width.

4. It is also preferable for most class intervals to have at least five occurrences; this may not be practical for the first and last intervals.

Example 4.3. Using these rules, the discharges for Mono Creek listed in Table 4.1 are placed into a frequency histogram using class intervals of 5 m^3/s (SI) and 200 ft^3/s (CU units) (see Table 4.4). These data can also be represented graphically by a frequency histogram as shown in Figure 4.6. Since relative frequency has been defined as the number of events in a certain class of events divided by the sample size, the histogram can also represent relative frequency (or probability) as shown on the right-hand ordinate of Figure 4.6.

From this frequency histogram, several features of the data can now be illustrated. Notice that there are some ranges of magnitudes that have occurred more frequently than others; also notice that the data are somewhat spread out and that the distribution of the ordinates is not symmetrical. While an effort was made to have frequencies of five or more, this was not possible with the class intervals selected. Because of the small sample size, it is difficult to assess the distribution of the population using the frequency histogram. It should also be noted that because the CU unit intervals are not a conversion from the SI, they represent an alternative interval selection. This illustrates that interval selection may influence the appearance of a histogram.

Table 4.4. Frequency Histogram and Relative Frequency Analysis of Annual Flood Data for Mono Creek

(a) 5 m^3/s intervals (SI)

Interval of Annual Floods (m^3/s)	Frequency	Relative Frequency	Cumulative Frequency
0 – 9.99	0	0.000	0.000
10 – 14.99	3	0.104	0.104
15 – 19.99	1	0.034	0.138
20 – 24.99	4	0.138	0.276
25 – 29.99	5	0.172	0.448
30 – 34.99	8	0.276	0.724
35 – 39.99	3	0.104	0.828
40 – 44.99	4	0.138	0.966
45 or larger	1	0.034	1.000

(b) 200 ft^3/s intervals (CU Units)

Interval of Annual Floods (ft^3/s)	Frequency	Relative Frequency	Cumulative Frequency
0 – 199	0	0.000	0.000
200 – 399	0	0.000	0.000
400 – 599	4	0.138	0.138
600 – 799	1	0.034	0.172
800 – 999	7	0.241	0.414
1000 – 1199	7	0.241	0.655
1200 – 1399	5	0.172	0.828
1400 – 1599	4	0.138	0.966
1600 – 1799	1	0.034	1.000

Example 4.4. Many flood records have relatively small record lengths. For such records, histograms may not be adequate to assess the shape characteristics of the distribution of floods. The flood record for Pond Creek of Table 4.3 provides a good illustration. With a record length of 24, it would be impractical to use more than 5 or 6 intervals when creating a histogram. Three histograms were compiled from the annual flood series (see Table 4.5). The first histogram uses an interval of 40 m^3/s (1,412 ft^3/s) and results in a hydrograph-like shape, with few values in the lowest cell and a noticeable peak in the second cell. The second histogram uses an interval of 50 m^3/s (1,766 ft^3/s). This produces a box-like shape with the first two cells having a large number of occurrences and the other cells very few, with one intermediate cell not having any occurrences. The third histogram uses an unequal cell width and produces an exponential-decay shape. These results indicate that short record lengths make it difficult to identify the distribution of floods.

Table 4.5. Alternative Frequency (f) Histograms of the Pond Creek, KY, Annual Maximum Flood Record (1945-1968)

Interval	Histogram 1 Frequency	Histogram 2 Frequency	Histogram 3 Frequency	**Histogram 3 Interval**	
				(m^3/s)	**(ft^3/s)**
1	3	10	10	0 – 50	0 – 1,765
2	13	10	5	50 – 75	1,766 – 2,648
3	4	3	5	75 – 100	2,649 – 3,531
4	3	0	3	100 – 150	3,532 – 5,297
5	1	1	1	> 150	> 5,297

4.2.7.2 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

$$\overline{Q} = \frac{\sum_{i=1}^{n} Q_i}{n} \qquad (4.12)$$

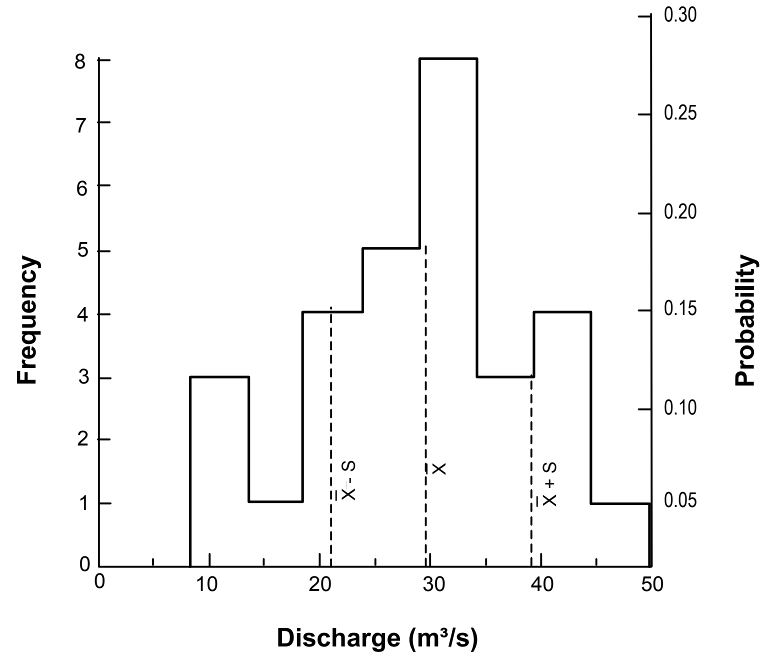


Figure 4.6a. Sample frequency histogram and probability, Mono Creek, CA ($\bar{X}$ = 30.0 m^3/s and S = 9.3 m^3/s)

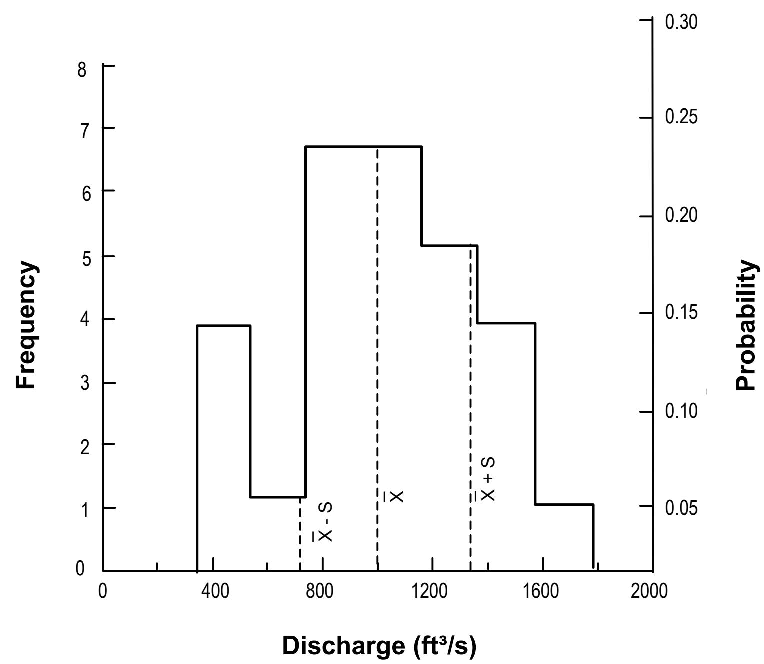


Figure 4.6b. Sample frequency histogram and probability, Mono Creek, CA ($\bar{X}$ = 1060 ft^3/s and S = 330 ft^3/s)

where,

$\overline{Q}$ = average or mean peak.

The median, another measure of central tendency, is the value of the middle item when the items are arranged according to magnitude. When there is an even number of items, the median is taken as the average of the two central values.

The mode is a third measure of central tendency. The mode is the most frequent or most common value that occurs in a set of data. For continuous variables, such as discharge rates, the mode is defined as the central value of the most frequent class interval.

4.2.7.3 Variability

The spread of the data is called dispersion. 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:

$$S = \left[\frac{\sum_{i=1}^{n} (Q_i - \overline{Q})^2}{n-1} \right]^{0.5} = \overline{Q} \left[\frac{\sum_{i=1}^{n} \left(\frac{Q_i}{\overline{Q}} - 1 \right)^2}{n-1} \right]^{0.5} \tag{4.13}$$

The second expression on the right-hand side of Equation 4.13 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 variation, V, or the standard deviation divided by the mean peak:

$$V = \frac{S}{\overline{Q}} \tag{4.14}$$

4.2.7.4 Skew

The symmetry of the frequency distribution, or more accurately the asymmetry, is called skew. One common measure of skew is the coefficient of skew, G. The skew coefficient is calculated by:

$$G = \frac{n \sum_{i=1}^{n} (Q_i - \overline{Q})^3}{(n-1)(n-2)\, S^3} = \frac{n \sum_{i=1}^{n} \left(\frac{Q_i}{\overline{Q}} - 1 \right)^3}{(n-1)(n-2)\, V^3} \tag{4.15}$$

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

Example 4.5. The computations below illustrate the computation of measures of central tendency, standard deviation, variance, and coefficient of skew for the Mono Creek frequency distribution shown in Figure 4.6 based on the data provided in Table 4.6. The mean value of the sample of floods is 30 m^3/s (1,060 ft^3/s), the standard deviation is 9.3 m^3/s (330 ft^3/s), and the coefficient of variation is 0.31. The coefficient of skew is –0.19, which indicates that the distribution is skewed negatively to the left. For the flow data in Table 4.6, the median value is 30.0 m^3/s (1,060 ft^3/s). Computed values of the mean and standard deviation are also identified in Figure 4.6.

Variable	Value in SI	Value in CU
$\overline{X} = \frac{\sum_{i=1}^{n} X_i}{n}$	$\frac{868.6}{29} = 30.0\ m^3/s$	$\frac{3066}{29} = 1058\ ft^3/s$
$S = \overline{X}\left[\frac{\sum_{i=1}^{n}\left(\frac{X_i}{\overline{X}} - 1\right)^2}{n-1}\right]^{0.5}$	$30.0\left[\frac{2.677}{28}\right]^{0.5} = 9.3\ m^3/s$	$1058\left[\frac{2.677}{28}\right]^{0.5} = 327\ ft^3/s$
$V = \frac{S}{\overline{X}}$	$\frac{9.3}{30.0} = 0.31$	$\frac{327}{1{,}058} = 0.31$
$G = \frac{n\sum_{i=1}^{n}\left(\frac{X_i}{\overline{X}} - 1\right)^3}{(n-1)(n-2)V^3}$	$\frac{29(-0.1448)}{28(27)(0.31)^3} = -0.19$	$\frac{29(-0.1448)}{28(27)(0.31)^3} = -0.19$

Table 4.6. Computation of Statistical Characteristics: Annual Maximum Flows for Mono Creek, CA

Year	Rank	Annual Maximum (m^3/s)	Annual Maximum (ft^3/s)	$[(X/\bar{X})]$	$[(X/\bar{X})-1]$	$[(X/\bar{X})-1]^2$	$[(X/\bar{X})-1]^3$
1938	1	49.8	1,760	1.664	0.664	0.441	0.2929
1943	2	40.8	1,440	1.362	0.362	0.131	0.0473
1927	3	40.2	1,420	1.343	0.343	0.117	0.0402
1932	4	40.2	1,420	1.343	0.343	0.117	0.0402
1941	5	40.2	1,420	1.343	0.343	0.117	0.0402
1922	6	39.4	1,390	1.314	0.314	0.099	0.0310
1945	7	38.8	1,370	1.295	0.295	0.087	0.0257
1933	8	38.2	1,350	1.276	0.276	0.076	0.0211
1935	9	34.8	1,230	1.163	0.163	0.027	0.0043
1937	10	34.3	1,210	1.144	0.144	0.021	0.0030
1942	11	33.1	1,170	1.106	0.106	0.011	0.0012
1940	12	32.0	1,130	1.068	0.068	0.005	0.0003
1928	13	31.4	1,110	1.049	0.049	0.002	0.0001
1950	14	31.2	1,100	1.040	0.040	0.002	0.0001
1925	15	30.0	1,060	1.002	0.002	0.000	0.0000
1936	16	30.0	1,060	1.002	0.002	0.000	0.0000
1926	17	29.2	1,030	0.974	-0.026	0.001	0.0000
1947	18	28.0	988	0.934	-0.066	0.004	-0.0003
1923	19	26.6	940	0.889	-0.111	0.012	-0.0014
1949	20	25.9	916	0.866	-0.134	0.018	-0.0024
1946	21	25.8	910	0.860	-0.140	0.019	-0.0027
1944	22	24.2	855	0.808	-0.192	0.037	-0.0070
1930	23	24.0	848	0.802	-0.198	0.039	-0.0078
1948	24	23.7	838	0.792	-0.208	0.043	-0.0090
1929	25	21.2	750	0.709	-0.291	0.085	-0.0246
1939	26	15.3	540	0.511	-0.489	0.240	-0.1173
1931	27	14.9	525	0.496	-0.504	0.254	-0.1277
1924	28	13.8	488	0.461	-0.539	0.290	-0.1562
1934	29	11.4	404	0.382	-0.618	0.382	-0.2361
	TOTAL	**868.4**	**30,672**			**2.677**	**-0.1449**

4.2.7.5 Generalized and Weighted Skew

Three methods are available for representing 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 (i.e., the skew coefficient computed from the actual data) may not be accurate if the sample size is small. In this case, USGS Bulletin 17B (1982) recommends use of a generalized skew coefficient determined from a map that shows isolines of generalized skew coefficients of the logarithms of annual maximum stream flows throughout the United States. A map of generalized skew is provided in Bulletin 17B. This map also gives average skew coefficients by one-degree quadrangles over most of the country.

Often the station skew and generalized skew can be combined to provide a better estimate for a given sample of flood data. USGS Bulletin 17B (1982) 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 MSEs, which are 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:

$$G_W = \frac{MSE_{\overline{G}}\,(G) + MSE_G\,(\overline{G})}{MSE_{\overline{G}} + MSE_G} \tag{4.16}$$

where,

G_W = weighted skew
G = station skew
$\overline{G}$ = generalized skew
MSE_G, $MSE_{\overline{G}}$ = mean-square errors for the station and generalized skews, respectively.

Equation 4.16 is based on the assumption that station and generalized skew are independent. If they are independent, the weighted estimate will have a lower variance than either the station or generalized skew.

When $\overline{G}$ is taken from the map of generalized skews in USGS Bulletin 17B (1982), $MSE_{\overline{G}}$ = 0.302. The value of MSE_G can be obtained from Table 4.7, which is from Bulletin 17B, or approximated by the equation:

$$MSE_G = 10^{\left(A - B\left[\log_{10}\left(n/10\right)\right]\right)} \tag{4.17a}$$

where n is the record length and

$$A = -0.33 + 0.08\left|G\right| \quad \text{for } \left|G\right| \leq 0.90 \tag{4.17b}$$

$$A = -0.52 + 0.30\left|G\right| \quad \text{for } \left|G\right| > 0.90 \tag{4.17c}$$

and

$$B = 0.94 - 0.26\left|G\right| \quad \text{for } \left|G\right| \leq 1.50 \tag{4.17d}$$

$$B = 0.55 \quad \text{for } \left|G\right| > 1.50 \tag{4.17e}$$

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.

Table 4.7. Summary of Mean Square Error of Station Skew a Function of Record Length and Station Skew

Skew	Record Length, N or H (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	1.160	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

4.2.8 Probability Distribution Functions

If the frequency histogram from a very large population of floods was 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 histogram would approach a smooth curve (see Figure 4.7) where the ordinate axis density units are the inverse of the abscissa units. This curve, which is called the probability density function, f(Q), encloses an area of 1.0 or:

$$\int_{-\infty}^{\infty} f(Q)dQ = 1 \tag{4.18}$$

The cumulative distribution function, F(Q), equals the area under the probability density function, f(Q), from -∞ to Q:

$$F(Q) = \int_{\infty}^{Q} f(Q)dQ \tag{4.18a}$$

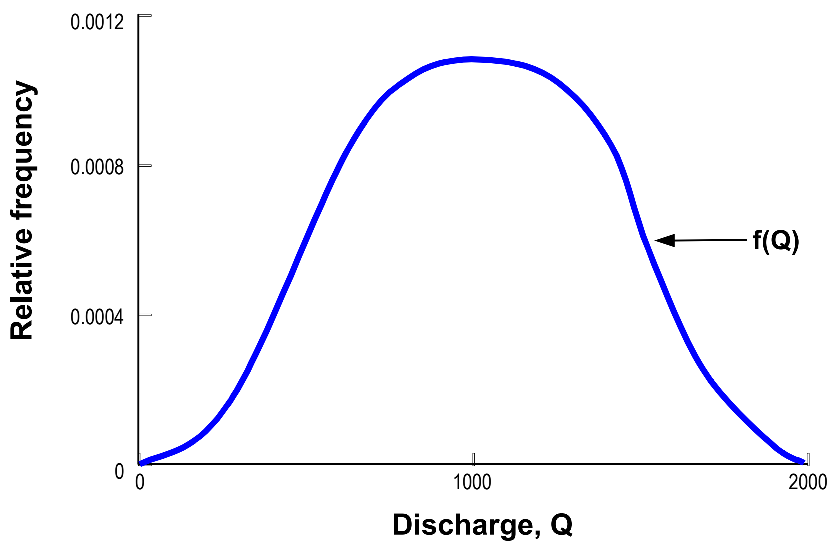


Figure 4.7. Probability density function

Equation 4.18 is a mathematical statement that the sum of the probabilities of all events is equal to unity. Two conditions of hydrologic probability are readily illustrated from Equations 4.18 and 4.18a. Figure 4.8a 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 4.8b shows the probability that a flood Q exceeds Q_1 is the area under the curve from Q_1 to infinity. From Equation 4.18a, this probability is given by $F(Q > Q_1) = 1 - F(Q < Q_1)$.

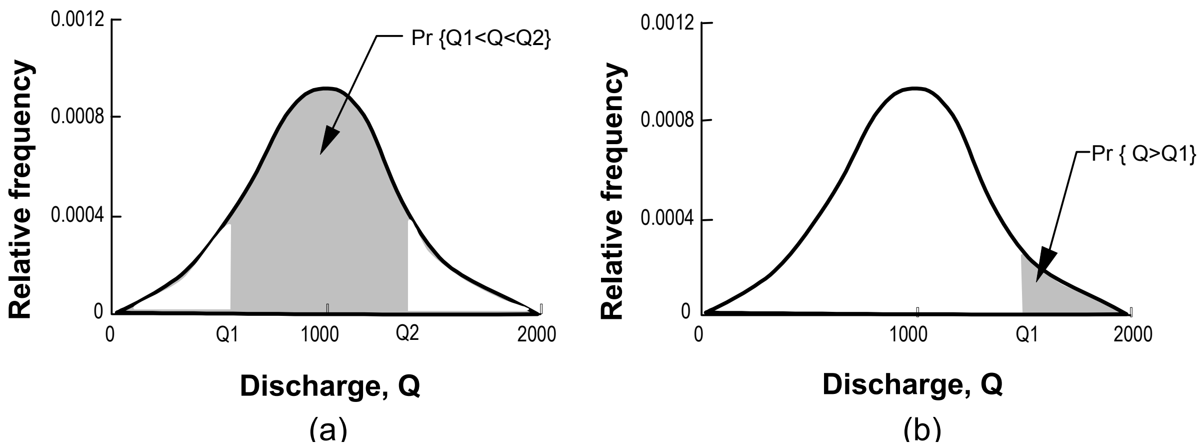


Figure 4.8. Hydrologic probability from density functions

As can be seen from Figure 4.8, the calculation for probability from the density function is somewhat tedious. A further refinement of the frequency distribution is the cumulative frequency distribution. Table 4.4 illustrates 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, Table 4.4 defines the number of times that floods equal or exceed the lower limit of the class interval and gives the cumulative frequency.

Using the cumulative frequency distribution, it is possible to compute directly the nonexceedence probability for a given magnitude. The nonexceedence probability is defined as the probability that the specified value will not be exceeded. The exceedence probability is 1.0 minus the nonexceedence probability. The sample cumulative frequency histogram for the Mono Creek, CA, annual flood series is shown in Figure 4.9.

Again, if the sample were very large so that small class intervals could be defined, the histogram becomes a smooth curve that is defined as the cumulative probability function, F(Q), shown in Figure 4.10a. This figure shows the area under the curve to the left of each Q of Figure 4.7 and defines the probability that the flow will be less than some stated value (i.e., the nonexceedence probability).

Another convenient representation for hydrologic analysis is the complementary probability function, G(Q), defined as:

$$G(Q) = 1 - F(Q) = P_r(Q \geq Q_1) \qquad (4.19)$$

The function, G(Q), shown in Figure 4.10b, is the exceedence probability (i.e., the probability that a flow of a given magnitude will be equaled or exceeded).

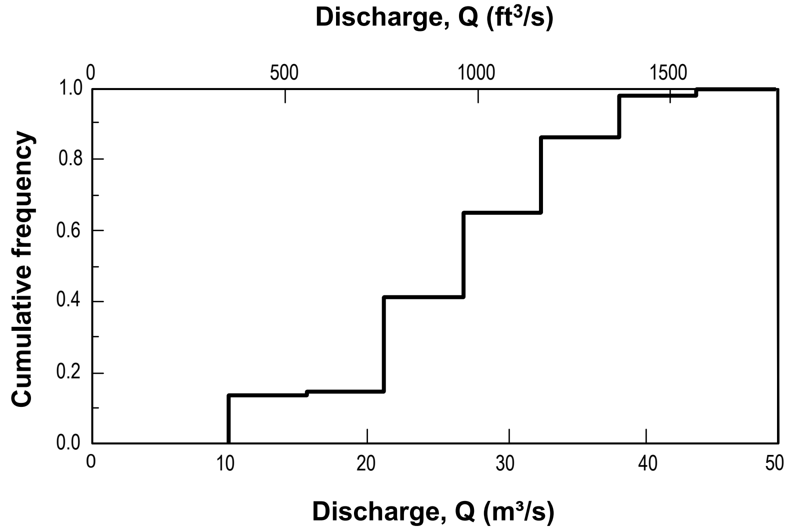


Figure 4.9. Cumulative frequency histogram, Mono Creek, CA

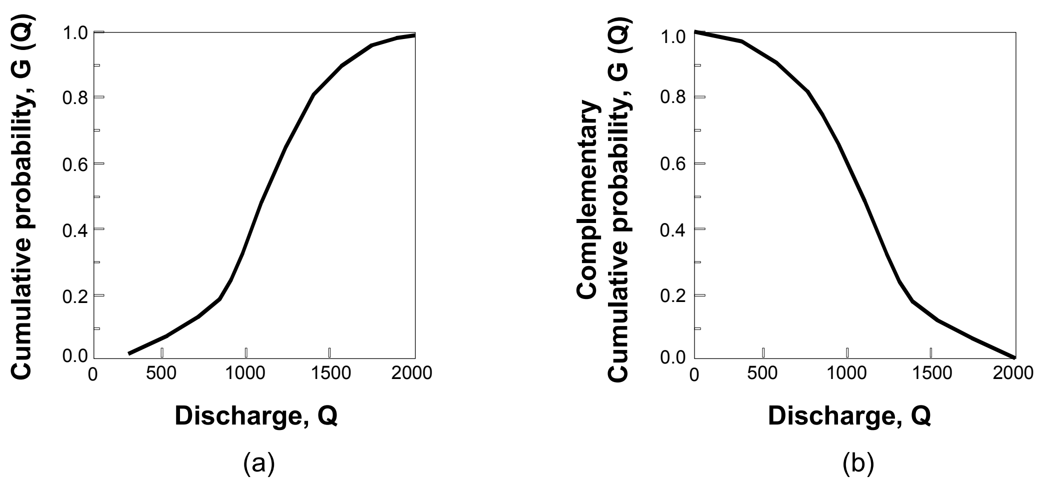


Figure 4.10. Cumulative and complementary cumulative distribution functions

4.2.9 Plotting Position Formulas

When making a flood frequency analysis, it is common to plot both the assumed population and the peak discharges of the sample. To plot the sample values on frequency paper, it is necessary to assign an exceedence probability to each magnitude. A plotting position formula is used for this purpose.

A number of different formulas have been proposed for computing plotting position probabilities, with no unanimity on the preferred method. Beard (1962) illustrates the nature of this problem. If a very long period of record, say 2,000 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 2,000-year flood, which is a flood with an exceedence probability of 0.0005. Some of the records will also contain 100-year floods and many will contain floods in excess of the true 20-year flood. Similarly some of the 20-year records will contain highest floods that are less than the true 20-year flood.

A general formula for computing plotting positions is:

$$P = \frac{i - a}{(n - a - b + 1)} \quad (4.20)$$

where,

- i = rank order of the ordered flood magnitudes, with the largest flood having a rank of 1
- n = record length
- a, b = constants for a particular plotting position formula.

The Weibull, P_w (a = b = 0), Hazen, P_h (a = b = 0.5), and Cunnane, P_c (a = b = 0.4) are three possible plotting position formulas:

$$P_w = \frac{i}{n + 1} \quad (4.21a)$$

$$P_h = \frac{i - 0.5}{n} \quad (4.21b)$$

$$P_c = \frac{i - 0.4}{n + 0.2} \quad (4.21c)$$

The data are plotted by placing a point for each value of the flood series at the intersection of the flood magnitude and the exceedence probability computed with the plotting position formula. The plotted data should approximate the population line if the assumed population model is a reasonable assumption.

For the partial-duration series where the number of floods exceeds the number of years of record, Beard (1962) recommends:

$$P = \frac{2i - 1}{2n} = \frac{i - 0.5}{n} \quad (4.22)$$

where i is the rank order number of the event and n is the record length.

4.3 STANDARD FREQUENCY DISTRIBUTIONS

Several cumulative frequency distributions are commonly used in the analysis of hydrologic data and, as a result, they have been studied extensively and are now standardized. The frequency distributions that have been found most useful in hydrologic data analysis are the normal distribution, the log-normal distribution, the Gumbel extreme value distribution, and 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 Normal Distribution

The normal or Gaussian distribution is a classical mathematical distribution commonly used in the analysis of natural phenomena. The normal distribution has a symmetrical, unbounded, bell-shaped curve with the maximum value at the central point and extending from $-\infty$ to $+\infty$. The normal distribution is shown in Figure 4.11a.

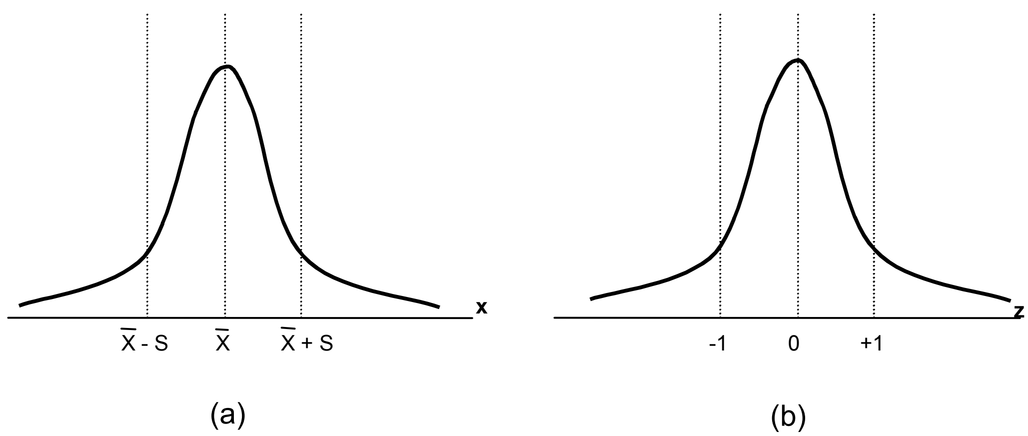


Figure 4.11. (a) Normal probability distribution; (b) Standard normal distribution

For the normal distribution, the maximum value occurs at the mean. Because of symmetry, half of the flows will be below the mean and half are above. Another characteristic of the normal distribution curve is that 68.3 percent of the events fall between ±1 standard deviation (S), 95 percent of the events fall within ±2S, and 99.7 percent fall within ±3S. In a sample of flows, these percentages will be approximated.

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

$$f(X) = \frac{e^{-\left[\left(x-\overline{x}\right)^2/2S^2\right]}}{S\sqrt{2\pi}} \tag{4.23}$$

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

One 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.

4.3.1.1 Standard Normal Distribution

A special case of the normal distribution of Equation 4.23 is called the standard normal distribution and is represented by the variate z (see Figure 4.11b). The standard normal distribution always has a mean of 0 and a standard deviation of 1. If the random variable X has a normal distribution with mean $\bar{X}$ and standard deviation S, values of X can be transformed so that they have a standard normal distribution using the following transformation:

$$z = \frac{X - \bar{X}}{S} \tag{4.24}$$

If $\bar{X}$, S, and z for a given frequency are known, then the value of X corresponding to the frequency can be computed by algebraic manipulation of Equation 4.24:

$$X = \bar{X} + zS \tag{4.25}$$

To illustrate, the 10-year event has an exceedence probability of 0.10 or a nonexceedence probability of 0.90. Thus, the corresponding value of z from Table 4.8 is 1.2816. If floods have a normal distribution with a mean of 120 m^3/s (4,240 ft^3/s) and a standard deviation of 35 m^3/s (1,230 ft^3/s), the 10-year flood for a normal distribution is computed with Equation 4.25:

Variable	Value in SI	Value in CU
$X = \bar{X} + zS$	$= 120 + 1.2816(35) = 165\ m^3/s$	$= 4240 + 1.2816(1230) = 165\ ft^3/s$

Similarly, the frequency of a flood of 181 m^3/s (6,390 ft^3/s) can be estimated using the transform of Equation 4.24:

Variable	Value in SI	Value in CU
$z = \frac{X - \bar{X}}{S}$	$= \frac{181 - 120}{35} = 1.75$	$= \frac{6390 - 4240}{1230} = 1.75$

From Table 4.8, this corresponds to an exceedence probability of 4 percent, which is the 25-year flood.

Table 4.8. Selected Values of the Standard Normal Deviate (z) for the Cumulative Normal Distribution

Exceedence Probability %	Return Period (yrs)	z
50	2	0.0000
20	5	0.8416
10	10	1.2816
4	25	1.7507
2	50	2.0538
1	100	2.3264
0.2	500	2.8782

4.3.1.2 Frequency Analysis for a Normal Distribution

An arithmetic-probability graph has a specially transformed horizontal probability scale. The horizontal scale is transformed in such a way that the cumulative distribution function for data that follow a normal distribution will plot as a straight line. If a series of peak flows that are normally distributed are plotted against the cumulative frequency function or the exceedence frequency on the probability scale, the data will plot as a straight line with the equation:

$$X = \overline{X} + K\,S \tag{4.26}$$

where X is the flood flow at a specified frequency. The value of K is the frequency factor of the distribution. For the normal distribution, K equals z where z is taken from Table 4.8.

The procedure for developing a frequency curve for the normal distribution is as follows:

1. Compute the mean $\overline{X}$ and standard deviation S of the annual flood series.

2. Plot two points on the probability paper: (a) $\overline{X}$ + S at an exceedence probability of 0.159 (15.9%) and (b) $\overline{X}$ - S at an exceedence probability of 0.841 (84.1%).

3. Draw a straight line through these two points; the accuracy of the graphing can be checked by ensuring that the line passes through the point defined by $\overline{X}$ at an exceedence probability of 0.50 (50%).

The straight line represents the assumed normal population. It can be used either to make probability estimates for given values of X or to estimate values of X for given exceedence probabilities.

4.3.1.3 Plotting Sample Data

Before a computed frequency curve is used to make estimates of either flood magnitudes or exceedence probabilities, the assumed population should be verified by plotting the data. The following steps are used to plot the data:

1. Rank the flood series in descending order, with the largest flood having a rank of 1 and the smallest flood having a rank of n.

2. Use the rank (i) with a plotting position formula such as Equation 4.21, and compute the plotting probabilities for each flood.

3. Plot the magnitude X against the corresponding plotting probability.

If the data follow the trend of the assumed population line, one usually assumes that the data are normally distributed. It is not uncommon for the sample points on the upper and lower ends to deviate from the straight line. Deciding whether or not to accept the computed straight line as the population is based on experience rather than an objective criterion.

4.3.1.4 Estimation with the Frequency Curve

Once the population line has been verified and accepted, the line can be used for estimation. While graphical estimates are acceptable for some work, it is often important to use Equations 4.24 and 4.25 in estimating flood magnitudes or probabilities. To make a probability estimate p for a given magnitude, use the following procedure:

1. Use Equation 4.24 to compute the value of the standard normal deviate.

2. Enter Table 4.9 with the value of z and obtain the exceedence probability.

To make estimates of the magnitude for a given exceedence probability, use the following procedure:

1. Enter Table 4.9 with the exceedence probability and obtain the corresponding value of z.

2. Use Equation 4.25 with $\overline{X}$, S, and z to compute the magnitude X.

Table 4.9. Probabilities of the Cumulative Standard Normal Distribution for Selected Values of the Standard Normal Deviate (z)

z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
-3.4	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0002
-3.3	0.0005	0.0005	0.0005	0.0004	0.0004	0.0004	0.0004	0.0004	0.0004	0.0003
-3.2	0.0007	0.0007	0.0006	0.0006	0.0006	0.0006	0.0006	0.0005	0.0005	0.0005
-3.1	0.0010	0.0009	0.0009	0.0009	0.0008	0.0008	0.0008	0.0008	0.0007	0.0007
-3.0	0.0013	0.0013	0.0013	0.0012	0.0012	0.0011	0.0011	0.0011	0.0010	0.0010
-2.9	0.0019	0.0018	0.0018	0.0017	0.0016	0.0016	0.0015	0.0015	0.0014	0.0014
-2.8	0.0026	0.0025	0.0024	0.0023	0.0023	0.0022	0.0021	0.0021	0.0020	0.0019
-2.7	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026
-2.6	0.0047	0.0045	0.0044	0.0043	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036
-2.5	0.0062	0.0060	0.0059	0.0057	0.0055	0.0054	0.0052	0.0051	0.0049	0.0048
-2.4	0.0082	0.0080	0.0078	0.0075	0.0073	0.0071	0.0069	0.0068	0.0066	0.0064
-2.3	0.0107	0.0104	0.0102	0.0099	0.0096	0.0094	0.0091	0.0089	0.0087	0.0084
-2.2	0.0139	0.0136	0.0132	0.0129	0.0125	0.0122	0.0119	0.0116	0.0113	0.0110
-2.1	0.0179	0.0174	0.0170	0.0166	0.0162	0.0158	0.0154	0.0150	0.0146	0.0143
-2.0	0.0228	0.0222	0.0217	0.0212	0.0207	0.0202	0.0197	0.0192	0.0188	0.0183
-1.9	0.0287	0.0281	0.0274	0.0268	0.0262	0.0256	0.0250	0.0244	0.0239	0.0233
-1.8	0.0359	0.0351	0.0344	0.0336	0.0329	0.0322	0.0314	0.0307	0.0301	0.0294
-1.7	0.0446	0.0436	0.0427	0.0418	0.0409	0.0401	0.0392	0.0384	0.0375	0.0367
-1.6	0.0548	0.0537	0.0526	0.0516	0.0505	0.0495	0.0485	0.0475	0.0465	0.0455
-1.5	0.0668	0.0655	0.0643	0.0630	0.0618	0.0606	0.0594	0.0582	0.0571	0.0559
-1.4	0.0808	0.0793	0.0778	0.0764	0.0749	0.0735	0.0721	0.0708	0.0694	0.0681
-1.3	0.0968	0.0951	0.0934	0.0918	0.0901	0.0885	0.0869	0.0853	0.0838	0.0823
-1.2	0.1151	0.1131	0.1112	0.1093	0.1075	0.1056	0.1038	0.1020	0.1003	0.0985
-1.1	0.1357	0.1335	0.1314	0.1292	0.1271	0.1251	0.1230	0.1210	0.1190	0.1170
-1.0	0.1587	0.1562	0.1539	0.1515	0.1492	0.1469	0.1446	0.1423	0.1401	0.1379
-.9	0.1841	0.1814	0.1788	0.1762	0.1736	0.1711	0.1685	0.1660	0.1635	0.1611
-.8	0.2119	0.2090	0.2061	0.2033	0.2005	0.1977	0.1949	0.1922	0.1894	0.1867
-.7	0.2420	0.2389	0.2358	0.2327	0.2296	0.2266	0.2236	0.2206	0.2177	0.2148
-.6	0.2743	0.2709	0.2676	0.2643	0.2611	0.2578	0.2546	0.2514	0.2483	0.2451
-.5	0.3085	0.3050	0.3015	0.2981	0.2946	0.2912	0.2877	0.2843	0.2810	0.2776
-.4	0.3446	0.3409	0.3372	0.3336	0.3300	0.3264	0.3228	0.3192	0.3156	0.3121
-.3	0.3821	0.3783	0.3745	0.3707	0.3669	0.3632	0.3594	0.3557	0.3520	0.3483
-.2	0.4207	0.4168	0.4129	0.4090	0.4052	0.4013	0.3974	0.3936	0.3897	0.3859
-.1	0.4602	0.4562	0.4522	0.4483	0.4443	0.4404	0.4364	0.4325	0.4286	0.4247
-.0	0.5000	0.4960	0.4920	0.4880	0.4840	0.4801	0.4761	0.4721	0.4681	0.4641

Table 4.9. Probabilities of the Cumulative Standard Normal Distribution for Selected Values of the Standard Normal Deviate (z)

z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7517	0.7549
0.7	0.7580	0.7611	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9864	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936
2.5	0.9938	0.9940	0.9941	0.9943	0.9945	0.9946	0.9948	0.9949	0.9951	0.9952
2.6	0.9953	0.9955	0.9956	0.9957	0.9959	0.9960	0.9961	0.9962	0.9963	0.9964
2.7	0.9965	0.9966	0.9967	0.9968	0.9969	0.9970	0.9971	0.9972	0.9973	0.9974
2.8	0.9974	0.9975	0.9976	0.9977	0.9977	0.9978	0.9979	0.9979	0.9980	0.9981
2.9	0.9981	0.9982	0.9982	0.9983	0.9984	0.9984	0.9985	0.9985	0.9986	0.9986
3.0	0.9987	0.9987	0.9987	0.9988	0.9988	0.9989	0.9989	0.9989	0.9990	0.9990
3.1	0.9990	0.9991	0.9991	0.9991	0.9992	0.9992	0.9992	0.9992	0.9993	0.9993
3.2	0.9993	0.9993	0.9994	0.9994	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995
3.3	0.9995	0.9995	0.9995	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9997
3.4	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9998

Example 4.6. To illustrate the use of these concepts, consider the data of Table 4.10. These data are the annual peak floods for the Medina River near San Antonio, Texas, for the period 1940-1982 (43 years of record) ranked from largest to smallest. Using Equations 4.12 and 4.13 for mean and standard deviation, respectively, and assuming the data are normally distributed, the 10-year and 100-year floods are computed as follows using SI and CU units:

Variable	Value in SI	Value in CU
$\overline{X} = \dfrac{\sum_{i=1}^{n} X_i}{n}$	$\dfrac{8{,}040}{43} = 187.0\ m^3/s$	$\dfrac{283{,}900}{43} = 6{,}602\ ft^3/s$
$S = \overline{X}\left[\dfrac{\sum_{i=1}^{n}\left(\dfrac{X_i}{\overline{X}} - 1\right)^2}{n - 1}\right]^{0.5}$	$187.0\left[\dfrac{48.22}{43-1}\right]^{0.5} = 200.4\ m^3/s$	$6{,}602\left[\dfrac{48.22}{43-1}\right]^{0.5} = 7{,}074\ ft^3/s$
$X_{10} = \overline{X} + z_{10}S$	$187.0 + 1.282\ (200.4)$ $= 444\ m^3/s$	$6{,}602 + 1.282\ (7{,}074)$ $= 15{,}700\ ft^3/s$
$X_{100} = \overline{X} + z_{100}S$	$187.0 + 2.326\ (200.4)$ $= 653\ m^3/s$	$6{,}602 + 2.326(7{,}074)$ $= 23{,}100\ ft^3/s$

When plotted on arithmetic probability scales, these two points are sufficient to establish the straight line on Figure 4.12 represented by Equation 4.26. For comparison, the measured discharges are plotted in Figure 4.12 using the Weibull plotting-position formula. The correspondence between the normal frequency curve and the actual data is poor. Obviously, the data are not normally distributed. Using Equations 4.14 and 4.15 to estimate the variance and skew, it becomes clear that the data have a large skew while the normal distribution has a skew of zero. This explains the poor correspondence in this case.

Variable	Value in SI	Value in CU
$V = \dfrac{S}{\overline{X}}$	$\dfrac{200.4}{187.0} = 1.072$	$\dfrac{7{,}074}{6{,}602} = 1.072$
$G = \dfrac{n\sum\left(\dfrac{X_i}{\overline{X}} - 1\right)^3}{(n - 1)(n - 2)V^3}$	$\dfrac{43(117.4)}{42(41)(1.072)^3} = 2.38$	$\dfrac{43(117.4)}{42(41)(1.072)^3} = 2.38$

Table 4.10. Frequency Analysis Computations for the Normal Distribution: Medina River, TX (Gage 08181500)

Year	Rank	Plotting Probability	Annual Maximum (m^3/s)	Annual Maximum (ft^3/s)	$X/\bar{X}$	$(X/\bar{X})-1$	$[(X/\bar{X})-1]^2$	$[(X-\bar{X})-1]^3$
1973	1	0.023	903.4	31,900	4.832	3.832	14.681	56.250
1946	2	0.045	900.6	31,800	4.816	3.816	14.565	55.586
1942	3	0.068	495.6	17,500	2.651	1.651	2.724	4.496
1949	4	0.091	492.8	17,400	2.635	1.635	2.674	4.374
1981	5	0.114	410.6	14,500	2.196	1.196	1.431	1.711
1968	6	0.136	371.0	13,100	1.984	0.984	0.968	0.953
1943	7	0.159	342.7	12,100	1.833	0.833	0.693	0.577
1974	8	0.182	274.1	9,680	1.466	0.466	0.217	0.101
1978	9	0.205	267.3	9,440	1.430	0.430	0.185	0.079
1958	10	0.227	261.1	9,220	1.396	0.396	0.157	0.062
1982	11	0.250	231.1	8,160	1.236	0.236	0.056	0.013
1976	12	0.273	212.7	7,510	1.137	0.137	0.019	0.003
1941	13	0.295	195.1	6,890	1.044	0.044	0.002	0.000
1972	14	0.318	180.1	6,360	0.963	-0.037	0.001	0.000
1950	15	0.341	160.3	5,660	0.857	-0.143	0.020	-0.003
1967	16	0.364	155.2	5,480	0.830	-0.170	0.029	-0.005
1965	17	0.386	153.8	5,430	0.822	-0.178	0.032	-0.006
1957	18	0.409	146.7	5,180	0.785	-0.215	0.046	-0.010
1953	19	0.432	140.5	4,960	0.751	-0.249	0.062	-0.015
1979	20	0.455	134.5	4,750	0.719	-0.281	0.079	-0.022
1977	21	0.477	130.8	4,620	0.700	-0.300	0.090	-0.027
1975	22	0.500	117.0	4,130	0.626	-0.374	0.140	-0.053
1962	23	0.523	112.1	3,960	0.600	-0.400	0.160	-0.064
1945	24	0.545	100.3	3,540	0.536	-0.464	0.215	-0.100
1970	25	0.568	95.2	3,360	0.509	-0.491	0.241	-0.118
1959	26	0.591	94.9	3,350	0.507	-0.493	0.243	-0.120
1960	27	0.614	90.6	3,200	0.485	-0.515	0.266	-0.137
1961	28	0.636	86.4	3,050	0.462	-0.538	0.289	-0.156
1971	29	0.659	83.5	2,950	0.447	-0.553	0.306	-0.169
1969	30	0.682	77.3	2,730	0.413	-0.587	0.344	-0.202
1940	31	0.705	71.9	2,540	0.385	-0.615	0.379	-0.233
1966	32	0.727	61.2	2,160	0.327	-0.673	0.453	-0.305
1951	33	0.750	60.9	2,150	0.326	-0.674	0.455	-0.307
1964	34	0.773	60.6	2,140	0.324	-0.676	0.457	-0.309
1948	35	0.795	58.1	2,050	0.310	-0.690	0.475	-0.328
1944	36	0.818	56.6	2,000	0.303	-0.697	0.486	-0.339
1980	37	0.841	56.1	1,980	0.300	-0.700	0.490	-0.343
1956	38	0.864	49.6	1,750	0.265	-0.735	0.540	-0.397
1947	39	0.886	41.6	1,470	0.223	-0.777	0.604	-0.470
1955	40	0.909	34.0	1,200	0.182	-0.818	0.670	-0.548
1963	41	0.932	25.2	890	0.135	-0.865	0.749	-0.648
1954	42	0.955	24.5	865	0.131	-0.869	0.755	-0.656
1952	43	0.977	22.7	801	0.121	-0.879	0.772	-0.679
Total			**8,040.3**	**283,906**			**48.22**	**117.4**

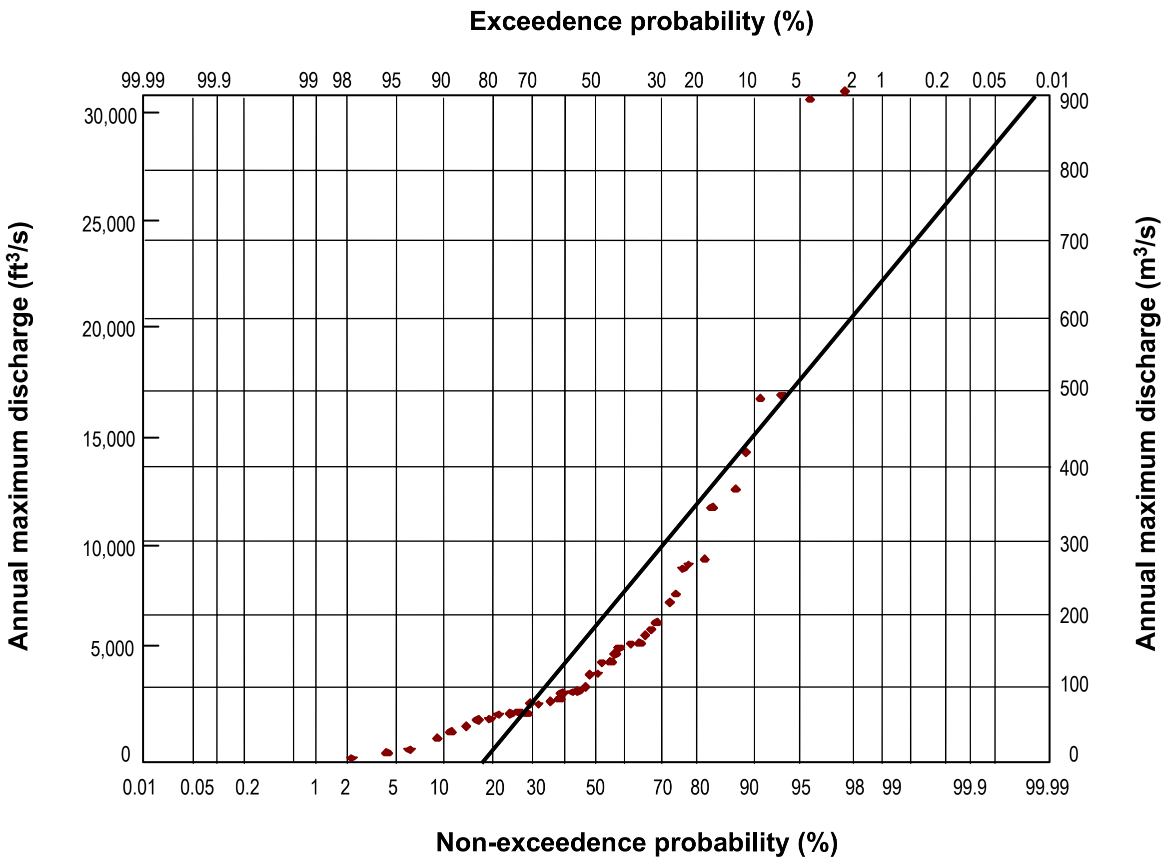


Figure 4.12. Normal distribution frequency curve, Medina River

4.3.2 Log-Normal Distribution

The log-normal distribution has the same characteristics as the normal distribution except that the dependent variable, X, 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 that result from an analysis of hydrologic data.

If a logarithmic transformation is performed on the normal distribution function, the resulting logarithmic distribution is normally distributed. This enables the z values tabulated in Tables 4-8 and 4-9 for a standard normal distribution to be used in a log-normal frequency analysis (Table 4.10). A three-parameter log-normal distribution exists, which makes use of a shift parameter. Only the zero-skew log-normal distribution will be discussed. As was the case with the normal distribution, log-normal probability scales have been developed, where the plot of the cumulative distribution function is a straight line. This scale uses 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 according to the equation:

$$Y = \log X = \overline{Y} + K\,S_y \tag{4.27}$$

where,

$\overline{Y}$ = average of the logarithms of X

S_y = standard deviation of the logarithms.

4.3.2.1 Procedure

The procedure for developing the graph of the log-normal distribution is similar to that for the normal distribution:

1. Transform the values of the flood series X by taking logarithms: Y = log X.

2. Compute the log mean ($\overline{Y}$) and log standard deviation (S_y) using the logarithms.

3. Using $\overline{Y}$ and S_y, compute $10^{\overline{Y} + S_y}$ and $10^{\overline{Y} - S_y}$. Using logarithmic frequency paper, plot these two values at exceedence probabilities of 0.159 (15.9%) and 0.841 (84.1%), respectively.

4. Draw a straight line through the two points.

The data points can now be plotted on the logarithmic probability paper using the same procedure as outlined for the normal distribution. Specifically, the flood magnitudes are plotted against the probabilities from a plotting position formula (e.g., Equation 4.21).

4.3.2.2 Estimation

Graphical estimates of either flood magnitudes or probabilities can be taken directly from the line representing the assumed log-normal distribution. Values can also be computed using either:

$$z = \frac{Y - \overline{Y}}{S_y} \tag{4.28}$$

to obtain a probability for the logarithm of a given magnitude (Y = log X) or:

$$Y = \overline{Y} + z\,S_y \tag{4.29}$$

to obtain a magnitude for a given probability. The value computed with Equation 4.29 must be transformed:

$$X = 10^Y \tag{4.30}$$

Two useful relations are also available to approximate the mean and the standard deviation of the logarithms, $\overline{Y}$ and S_y, from $\overline{X}$ and S of the original variables. These equations are

$$\overline{Y} = 0.5 \log\left(\frac{\overline{X}^4}{\overline{X}^2 + S^2}\right) \tag{4.31}$$

and

$$S_y = \left[log \left(\frac{S^2 + \overline{X}^2}{\overline{X}^2} \right) \right]^{0.5} \qquad (4.32)$$

<u>Example 4.7</u>. The log-normal distribution will be illustrated using the 43-year record from the Medina River shown in Table 4.11. Mean and standard deviation are calculated as follows:

Variable	Value in SI	Value in CU
$\overline{Y} = \frac{\sum_{i=1}^{n} Y_i}{n}$	$= \frac{89.92}{43} = 2.091$	$= \frac{156.48}{43} = 3.639$
$S_y = \overline{Y} \left[\frac{\sum_{i=1}^{n} \left(\frac{Y_i}{\overline{Y}} - 1 \right)^2}{n - 1} \right]^{0.5}$	$= 2.091 \left(\frac{1.492}{42} \right)^{0.5} = 0.394$	$= 3.639 \left(\frac{0.493}{42} \right)^{0.5} = 0.394$

Assuming the distribution of the logs is normal, the 10-year and 100-year floods are:

Variable	Value in SI	Value in CU
$Y_{10} = \overline{Y} + z_{10} S_y$	$=2.091+1.282\ (0.394)=2.596$	$=3.639+1.282\ (0.394)=4.144$
$X_{10} = 10^{Y_{10}}$	$= 10^{2.596} = 394\ m^3/s$	$= 10^{4.144} = 13{,}900\ ft^3/s$
$Y_{100} = \overline{Y} + z_{100} S_y$	$=2.091+2.326(0.394)=3.007$	$=3.639+2.326(0.394)=4.555$
$X_{100} = 10^{Y_{100}}$	$= 10^{3.007} = 1{,}020\ m^3/s$	$= 10^{4.555} = 35{,}900\ ft^3/s$

The measured flood data are also plotted on log-probability scales in Figure 4.13 together with the fitted log-normal distribution. (Note: When plotting X on the log scale, the actual values of X are plotted rather than their logarithms since the log-scale effectively transforms the data to their respective logarithms.) Figure 4.13 shows that the log-normal distribution fits the actual data better than the normal distribution shown in Figure 4.12. A smaller skew, as calculated below, explains the improved fit:

Variable	Value in SI	Value in CU
$V_y = \frac{S_y}{\overline{Y}}$	$= \frac{0.394}{2.091} = 0.188$	$= \frac{0.394}{3.639} = 0.108$
$G_y = \frac{n \sum_{i=1}^{n} \left(\frac{Y_i}{\overline{Y}} - 1 \right)^3}{(n-1)(n-2) V_y^3}$	$= \frac{43(0.06321)}{(42)(41)(0.188)^3} = 0.24$	$= \frac{43(0.01199)}{(42)(41)(0.108)^3} = 0.24$

Table 4.11. Frequency Analysis Computations for the Log-Normal Distribution: Medina River

(a) SI Units

Year	Rank	Plotting Probability	Annual Max.(X) (m^3/s)	Y = log(X)	$Y/\bar{Y}$	$[(Y/\bar{Y})-1]$	$[(Y/\bar{Y})-1]^2$	$[(Y/\bar{Y})-1]^3$
1973	1	0.023	903.4	2.956	1.413	0.413	0.1709	0.0707
1946	2	0.045	900.6	2.955	1.413	0.413	0.1704	0.0703
1942	3	0.068	495.6	2.695	1.289	0.289	0.0834	0.0241
1949	4	0.091	492.8	2.693	1.288	0.288	0.0827	0.0238
1981	5	0.114	410.6	2.613	1.250	0.250	0.0624	0.0156
1968	6	0.136	371.0	2.569	1.229	0.229	0.0523	0.0120
1943	7	0.159	342.7	2.535	1.212	0.212	0.0450	0.0095
1974	8	0.182	274.1	2.438	1.166	0.166	0.0275	0.0046
1978	9	0.205	267.3	2.427	1.161	0.161	0.0258	0.0041
1958	10	0.227	261.1	2.417	1.156	0.156	0.0242	0.0038
1982	11	0.250	231.1	2.364	1.130	0.130	0.0170	0.0022
1976	12	0.273	212.7	2.328	1.113	0.113	0.0128	0.0014
1941	13	0.295	195.1	2.290	1.095	0.095	0.0091	0.0009
1972	14	0.318	180.1	2.256	1.079	0.079	0.0062	0.0005
1950	15	0.341	160.3	2.205	1.054	0.054	0.0030	0.0002
1967	16	0.364	155.2	2.191	1.048	0.048	0.0023	0.0001
1965	17	0.386	153.8	2.187	1.046	0.046	0.0021	0.0001
1957	18	0.409	146.7	2.166	1.036	0.036	0.0013	0.0000
1953	19	0.432	140.5	2.148	1.027	0.027	0.0007	0.0000
1979	20	0.455	134.5	2.129	1.018	0.018	0.0003	0.0000
1977	21	0.477	130.8	2.117	1.012	0.012	0.0001	0.0000
1975	22	0.500	117.0	2.068	0.989	-0.011	0.0001	0.0000
1962	23	0.523	112.1	2.050	0.980	-0.020	0.0004	0.0000
1945	24	0.545	100.3	2.001	0.957	-0.043	0.0019	-0.0001
1970	25	0.568	95.2	1.978	0.946	-0.054	0.0029	-0.0002
1959	26	0.591	94.9	1.977	0.945	-0.055	0.0030	-0.0002
1960	27	0.614	90.6	1.957	0.936	-0.064	0.0041	-0.0003
1961	28	0.636	86.4	1.936	0.926	-0.074	0.0055	-0.0004
1971	29	0.659	83.5	1.922	0.919	-0.081	0.0066	-0.0005
1969	30	0.682	77.3	1.888	0.903	-0.097	0.0094	-0.0009
1940	31	0.705	71.9	1.857	0.888	-0.112	0.0126	-0.0014
1966	32	0.727	61.2	1.787	0.854	-0.146	0.0212	-0.0031
1951	33	0.750	60.9	1.785	0.853	-0.147	0.0215	-0.0032
1964	34	0.773	60.6	1.783	0.852	-0.148	0.0218	-0.0032
1948	35	0.795	58.1	1.764	0.843	-0.157	0.0245	-0.0038
1944	36	0.818	56.6	1.753	0.838	-0.162	0.0261	-0.0042
1980	37	0.841	56.1	1.749	0.836	-0.164	0.0268	-0.0044
1956	38	0.864	49.6	1.695	0.811	-0.189	0.0359	-0.0068
1947	39	0.886	41.6	1.619	0.774	-0.226	0.0509	-0.0115
1955	40	0.909	34.0	1.531	0.732	-0.268	0.0717	-0.0192
1963	41	0.932	25.2	1.401	0.670	-0.330	0.1088	-0.0359
1954	42	0.955	24.5	1.389	0.664	-0.336	0.1127	-0.0378
1952	43	0.977	22.7	1.355	0.648	-0.352	0.1239	-0.0436
Total			**8,040.3**	**89.92**			**1.992**	**0.06321**

Table 4.11. Frequency Analysis Computations for the Log-Normal Distribution: Medina River (Continued)

(b) CU Units

Year	Rank	Plotting Probability	Annual Max.(x) (ft^3/s)	Y = Log(X)	$Y/\bar{Y}$	$[(Y/\bar{Y})-1]$	$[(Y/\bar{Y})-1]^2$	$[(Y/\bar{Y})-1]^3$
1973	1	0.023	31,900	4.504	1.238	0.238	0.0565	0.0134
1946	2	0.045	31,800	4.502	1.237	0.237	0.0563	0.0133
1942	3	0.068	17,500	4.243	1.166	0.166	0.0275	0.0046
1949	4	0.091	17,400	4.241	1.165	0.165	0.0273	0.0045
1981	5	0.114	14,500	4.161	1.144	0.144	0.0206	0.0030
1968	6	0.136	13,100	4.117	1.131	0.131	0.0173	0.0023
1943	7	0.159	12,100	4.083	1.122	0.122	0.0149	0.0018
1974	8	0.182	9,680	3.986	1.095	0.095	0.0091	0.0009
1978	9	0.205	9,440	3.975	1.092	0.092	0.0085	0.0008
1958	10	0.227	9,220	3.965	1.089	0.089	0.0080	0.0007
1982	11	0.250	8,160	3.912	1.075	0.075	0.0056	0.0004
1976	12	0.273	7,510	3.876	1.065	0.065	0.0042	0.0003
1941	13	0.295	6,890	3.838	1.055	0.055	0.0030	0.0002
1972	14	0.318	6,360	3.803	1.045	0.045	0.0020	0.0001
1950	15	0.341	5,660	3.753	1.031	0.031	0.0010	0.0000
1967	16	0.364	5,480	3.739	1.027	0.027	0.0007	0.0000
1965	17	0.386	5,430	3.735	1.026	0.026	0.0007	0.0000
1957	18	0.409	5,180	3.714	1.021	0.021	0.0004	0.0000
1953	19	0.432	4,960	3.695	1.015	0.015	0.0002	0.0000
1979	20	0.455	4,750	3.677	1.010	0.010	0.0001	0.0000
1977	21	0.477	4,620	3.665	1.007	0.007	0.0000	0.0000
1975	22	0.500	4,130	3.616	0.994	-0.006	0.0000	0.0000
1962	23	0.523	3,960	3.598	0.989	-0.011	0.0001	0.0000
1945	24	0.545	3,540	3.549	0.975	-0.025	0.0006	0.0000
1970	25	0.568	3,360	3.526	0.969	-0.031	0.0010	0.0000
1959	26	0.591	3,350	3.525	0.969	-0.031	0.0010	0.0000
1960	27	0.614	3,200	3.505	0.963	-0.037	0.0014	0.0000
1961	28	0.636	3,050	3.484	0.957	-0.043	0.0018	-0.0001
1971	29	0.659	2,950	3.470	0.953	-0.047	0.0022	-0.0001
1969	30	0.682	2,730	3.436	0.944	-0.056	0.0031	-0.0002
1940	31	0.705	2,540	3.405	0.936	-0.064	0.0041	-0.0003
1966	32	0.727	2,160	3.334	0.916	-0.084	0.0070	-0.0006
1951	33	0.750	2,150	3.332	0.916	-0.084	0.0071	-0.0006
1964	34	0.773	2,140	3.330	0.915	-0.085	0.0072	-0.0006
1948	35	0.795	2,050	3.312	0.910	-0.090	0.0081	-0.0007
1944	36	0.818	2,000	3.301	0.907	-0.093	0.0086	-0.0008
1980	37	0.841	1,980	3.297	0.906	-0.094	0.0089	-0.0008
1956	38	0.864	1,750	3.243	0.891	-0.109	0.0118	-0.0013
1947	39	0.886	1,470	3.167	0.870	-0.130	0.0168	-0.0022
1955	40	0.909	1,200	3.079	0.846	-0.154	0.0237	-0.0036
1963	41	0.932	890	2.949	0.810	-0.190	0.0359	-0.0068
1954	42	0.955	865	2.937	0.807	-0.193	0.0372	-0.0072
1952	43	0.977	801	2.903	0.798	-0.202	0.0409	-0.0083
Total			**283,906**	**156.48**			**0.492**	**0.0121**

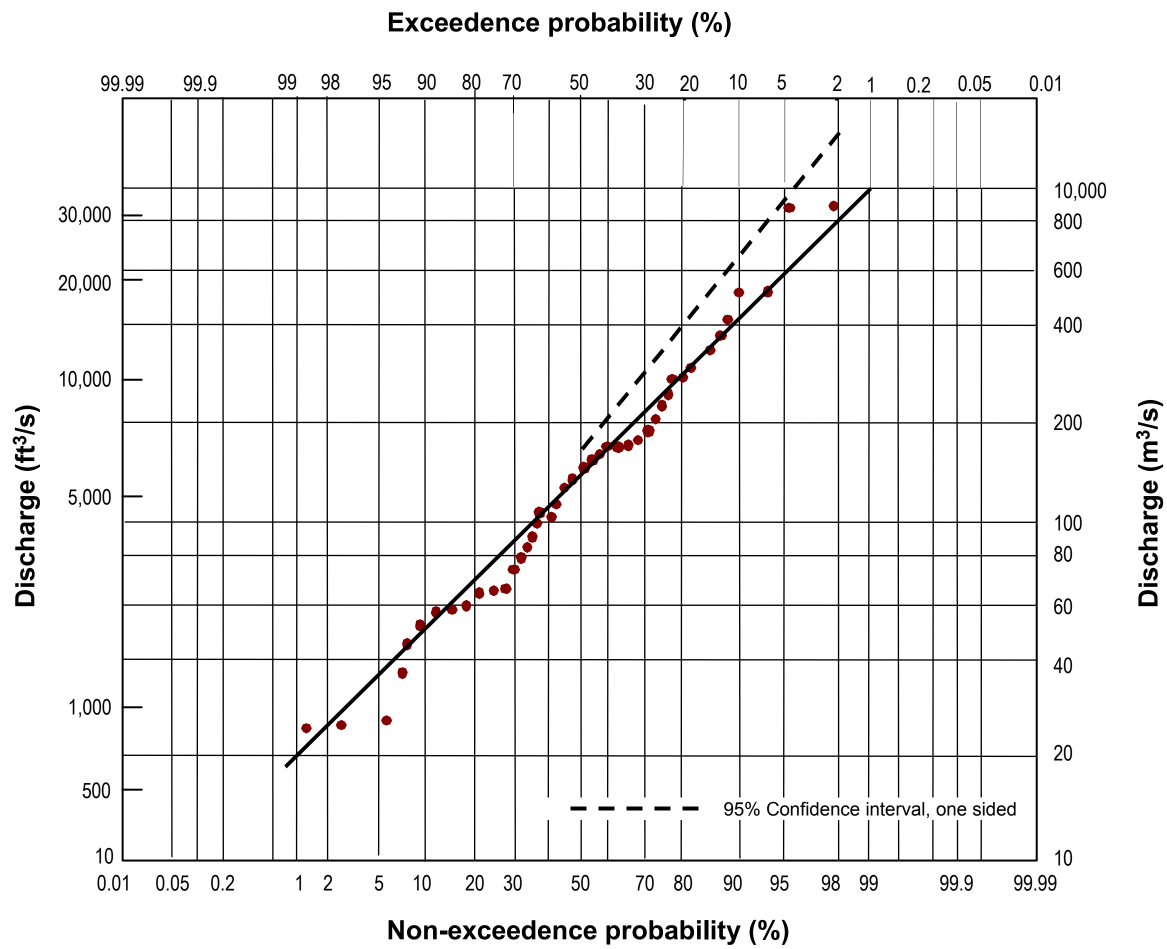


Figure 4.13. Log-normal distribution frequency curve (solid line) and one-sided upper confidence interval (dashed line)

4.3.3 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:

$$F(X) = e^{-e^{\alpha(X-\beta)}} \tag{4.33}$$

where,

$$\alpha = \frac{1.281}{S} \tag{4.33a}$$

$$\beta = \overline{X} - 0.450\ S \tag{4.33b}$$

In a manner analogous to that of the normal distribution, values of the distribution function can be computed from Equation 4.33. Frequency factor values K are tabulated for convenience in Table 4.12 for use in Equation 4.26.

Table 4.12. Frequency Factors (K) for the Gumbel Extreme Value Distribution

Sample Size n	Exceedence 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

Characteristics of the Gumbel extreme-value distribution are that the mean flow, $\bar{X}$, occurs at the return period of T_r = 2.33 years and that it has a positive skew (i.e., it is skewed toward the high flows or extreme values).

As was the case with the two previous distributions, special probability scales have been developed so that sample data, if they are distributed according to Equation 4.33, will plot as a straight line. Most USGS offices have prepared forms with these axis on which the horizontal scale has been transformed by the double-logarithmic transform of Equation 4.33.

<u>Example 4.8</u>. Peak flow data for the Medina River can be fit with a Gumbel distribution using Equation 4.26 and values of K from Table 4.12. The mean and standard deviation were calculated earlier as 187.0 m^3/s (6,602 ft^3/s) and 200.4 m^3/s (7,074 ft^3/s), respectively. The 10-year flood computed from the Gumbel distribution is:

Variable	Value in SI	Value in CU
$X_{10} = \bar{X} + KS$	$187.0 + 1.486\ (200.4) = 485\ m^3/s$	$6{,}602 + 1.486\ (7{,}074) = 17{,}100\ ft^3/s$

and the 100-year flood is:

Variable	Value in SI	Value in CU
$X_{100} = \bar{X} + KS$	$187.0 + 3.534\ (200.4) = 895\ m^3/s$	$6{,}602 + 3.534\ (7{,}074) = 31{,}600\ ft^3/s$

Plotted on the Gumbel graph in Figure 4.14 are the actual flood data and the computed frequency curve.

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 4.33.

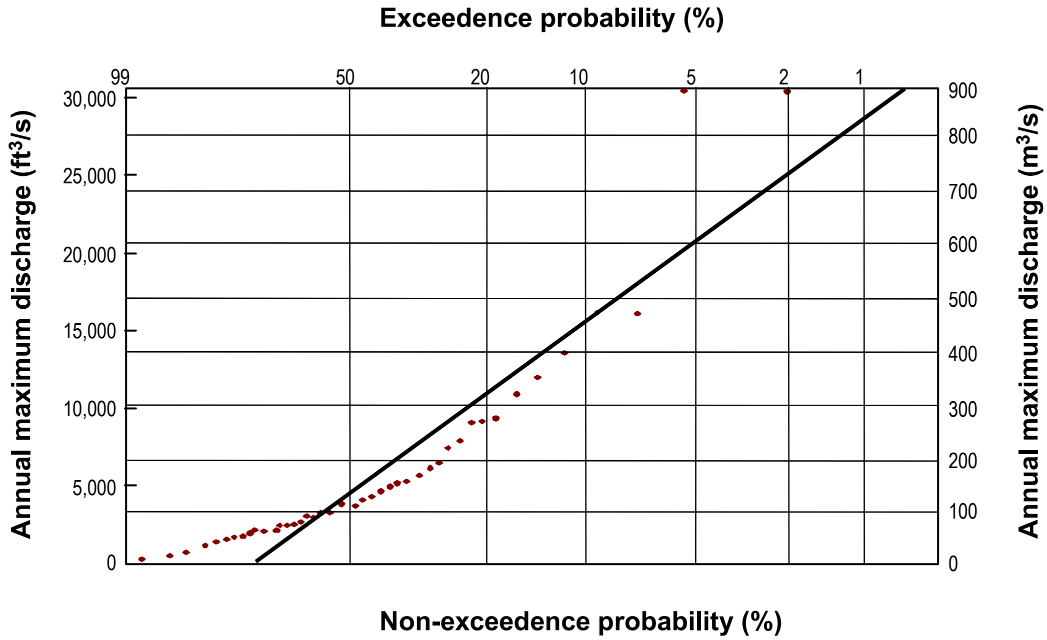


Figure 4.14. Gumbel extreme value distribution frequency curve, Medina River

4.3.4 Log-Pearson Type III Distribution

Another distribution that 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 variable. It is widely used for flood analyses because the data quite frequently fit the assumed population. It is this flexibility that led the Interagency Advisory Committee on Water Data to recommend its use as the standard distribution for flood frequency studies by all U.S. Government agencies. Thomas (1985) describes the motivation for adopting the log-Pearson Type III distribution and the events leading up to USGS Bulletin 17B (1982).

The log-Pearson Type III distribution differs from most of the distributions discussed above in that three parameters (mean, 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 shape of distribution. An extensive treatment on the use of this distribution in the determination of flood frequency distributions is presented in USGS Bulletin 17B, "Guidelines for Determining Flood Frequency" by the Interagency Advisory Committee on Water Data, revised March 1982. The Bulletin 17B procedure assumes the logarithms of the annual peak flows are Pearson Type III distributed rather than assuming the untransformed data are log-Pearson Type III. Kite (1988) has a good description of the two approaches.

An abbreviated table of the log-Pearson Type III distribution function is given in Table 4.13. (Extensive tables that reduce the amount of interpolation can be found in USGS Bulletin 17B, 1982.) Using the mean, standard deviation, and skew coefficient for any set of log-transformed annual peak flow data, in conjunction with Table 4.13, the flood with any exceedence frequency can be computed from the equation:

$$\hat{Y} = \log X = \overline{Y} + KS_y \tag{4.34}$$

where $\hat{Y}$ is the predicted value of log X, $\overline{Y}$ and S_y are as previously defined, and K is a function of the exceedence probability and the coefficient of skew.

Again, it would be possible to develop special probability scales, so that the log-Pearson Type III distribution would plot as a straight line. However, the log-Pearson Type III distribution can assume a variety of shapes so that a separate probability scale would be required for each different shape. Since this is impractical, log-Pearson Type III distributions are usually plotted on log-normal probability scales even though the plotted frequency distribution may not be a straight line. It is a straight line only when the skew of the logarithms is zero.

4.3.4.1 Procedure

The procedure for fitting the log-Pearson Type III distribution is similar to that for the normal and log-normal. The specific steps for making a basic log-Pearson Type III analysis without any of the optional adjustments are as follows:

1. Make a logarithmic transform of all flows in the series ($Y_i = \log X_i$).

2. Compute the mean ($\overline{Y}$), standard deviation (S_y), and standardized skew (G) of the logarithms using Equations 4.12, 4.13, and 4.15, respectively. Round the skew to the nearest tenth (e.g., 0.32 is rounded to 0.3).

3. Since the log-Pearson Type III curve with a nonzero skew does not plot as a straight line, it is necessary to use more than two points to draw the curve. The curvature of the line will increase as the absolute value of the skew increases, so more points will be needed for larger skew magnitudes.

4. Compute the logarithmic value $\hat{Y}$ for each exceedence frequency using Equation 4.34.

5. Transform the computed values of step 4 to discharges using equation 4.35:

$$\hat{X} = 10^{\hat{Y}} \tag{4.35}$$

   in which $\hat{X}$ is the computed discharge for the assumed log-Pearson Type III population.

6. Plot the points of step 5 on logarithmic probability paper and draw a smooth curve through the points.

The sample data can be plotted on the paper using a plotting position formula to obtain the exceedence probability. The computed curve can then be verified, and, if acceptable, it can be used to make estimates of either a flood probability or flood magnitude.

4.3.4.2 Estimation

In addition to graphical estimation, estimates can be made with the mathematical model of Equation 4.34. To compute a magnitude for a given probability, the procedure is the same as that in steps 3 to 5 above. To estimate the probability for a given magnitude X, the value is transformed using the logarithm (Y = log X) and then Equation 4.34 is algebraically transformed to compute K:

$$K = \frac{Y - \overline{Y}}{S_y} \tag{4.36}$$

The computed value of K should be compared to the K values of Table 4.13 for the standardized skew and a value of the probability interpolated from the probability values on Table 4.13; linear interpolation is acceptable.

Example 4.9. The log-Pearson Type III distribution will be illustrated using the Medina River flood data (Table 4.11). Three cases will be computed: station skew, generalized skew, and weighted skew. Table 4.13 and Equation 4.34 are used to compute values of the log-Pearson Type III distribution for the 10- and 100-year flood using the parameters, $\overline{Y}$, S_y, and G for the Medina River flood data. (To help define the distribution, the 2-, 5-, 25-, and 50-year floods have also been computed in Table 4.14.) Rounding the station skew of 0.236 to 0.2, the log-Pearson Type III distribution estimates of the 100- and 10-year floods are 1,160 m^3/s (41,000 ft^3/s) and 402 m^3/s (14,200 ft^3/s), respectively. The log-Pearson Type III distribution (G = 0.2) and the actual data from Table 4.11 are plotted in Figure 4.15 on log-normal probability scales.

The generalized skew coefficient for the Medina River is -0.252, which can be rounded to -0.3. Using this option, the 10- and 100-year floods for the Medina River are estimated as shown in Table 4.15. This log-Pearson Type III distribution (generalized skew coefficient, $\overline{G}$ = -0.3) is also plotted on Figure 4.15.

To illustrate the use of weighted skew, the station and generalized skews have already been determined to be G = 0.236 and $\overline{G}$ = -0.252, respectively. The mean-square error of $\overline{G}$, $MSE_{\overline{G}}$, is 0.302 and from Equation 4.17, MSE_G = 0.136. From Equation 4.16, the weighted skew is:

$$G_w = \frac{0.302(0.236) + 0.136(-0.252)}{0.302 + 0.136} = 0.084$$

which is rounded to 0.1 when obtaining values from Table 4.13. Values for selected return periods are given in Table 4.16.

Table 4.13. Frequency Factors (K) for the Log-Pearson Type III Distribution

	Skew						
Prob.	**-2.0**	**-1.9**	**-1.8**	**-1.7**	**-1.6**	**-1.5**	**-1.4**
0.9999	-8.21034	-7.98888	-7.76632	-7.54272	-7.31818	-7.09277	-6.86661
0.9995	-6.60090	-6.44251	-6.28285	-6.12196	-5.95990	-5.79673	-5.63252
0.9990	-5.90776	-5.77549	-5.64190	-5.50701	-5.37087	-5.23353	-5.09505
0.9980	-5.21461	-5.10768	-4.99937	-4.88971	-4.77875	-4.66651	-4.55304
0.9950	-4.29832	-4.22336	-4.14700	-4.06926	-3.99016	-3.90973	-3.82798
0.9900	-3.60517	-3.55295	-3.49935	-3.44438	-3.38804	-3.33035	-3.27134
0.9800	-2.91202	-2.88091	-2.84848	-2.81472	-2.77964	-2.74325	-2.70556
0.9750	-2.68888	-2.66413	-2.63810	-2.61076	-2.58214	-2.55222	-2.52102
0.9600	-2.21888	-2.20670	-2.19332	-2.17873	-2.16293	-2.14591	-2.12768
0.9500	-1.99573	-1.98906	-1.98124	-1.97227	-1.96213	-1.95083	-1.93836
0.9000	-1.30259	-1.31054	-1.31760	-1.32376	-1.32900	-1.33330	-1.33665
0.8000	-0.60944	-0.62662	-0.64335	-0.65959	-0.67532	-0.69050	-0.70512
0.7000	-0.20397	-0.22250	-0.24094	-0.25925	-0.27740	-0.29535	-0.31307
0.6000	0.08371	0.06718	0.05040	0.03344	0.01631	-0.00092	-0.01824
0.5704	0.15516	0.13964	0.12381	0.10769	0.09132	0.07476	0.05803
0.5000	0.30685	0.29443	0.28150	0.26808	0.25422	0.23996	0.22535
0.4296	0.43854	0.43008	0.42095	0.41116	0.40075	0.38977	0.37824
0.4000	0.48917	0.48265	0.47538	0.46739	0.45873	0.44942	0.43949
0.3000	0.64333	0.64453	0.64488	0.64436	0.64300	0.64080	0.63779
0.2000	0.77686	0.78816	0.79868	0.80837	0.81720	0.82516	0.83223
0.1000	0.89464	0.91988	0.94496	0.96977	0.99418	1.01810	1.04144
0.0500	0.94871	0.98381	1.01973	1.05631	1.09338	1.13075	1.16827
0.0400	0.95918	0.99672	1.03543	1.07513	1.11566	1.15682	1.19842
0.0250	0.97468	1.01640	1.06001	1.10537	1.15229	1.20059	1.25004
0.0200	0.97980	1.02311	1.06864	1.11628	1.16584	1.21716	1.26999
0.0100	0.98995	1.03695	1.08711	1.14042	1.19680	1.25611	1.31815
0.0050	0.99499	1.04427	1.09749	1.15477	1.21618	1.28167	1.35114
0.0020	0.99800	1.04898	1.10465	1.16534	1.23132	1.30279	1.37981
0.0010	0.99900	1.05068	1.10743	1.16974	1.23805	1.31275	1.39408
0.0005	0.99950	1.05159	1.10901	1.17240	1.24235	1.31944	1.40413
0.0001	0.99990	1.05239	1.11054	1.17520	1.24728	1.32774	1.41753

Table 4.13. Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

Prob.	Skew						
	-1.3	-1.2	-1.1	-1.0	-0.9	-0.8	-0.7
0.9999	-6.63980	-6.41249	-6.18480	-5.95691	-5.72899	-5.50124	-5.27389
0.9995	-5.46735	-5.30130	-5.13449	-4.96701	-4.79899	-4.63057	-4.46189
0.9990	-4.95549	-4.81492	-4.67344	-4.53112	-4.38807	-4.24439	-4.10022
0.9980	-4.43839	-4.32263	-4.20582	-4.08802	-3.96932	-3.84981	-3.72957
0.9950	-3.74497	-3.66073	-3.57530	-3.48874	-3.40109	-3.31243	-3.22281
0.9900	-3.21103	-3.14944	-3.08660	-3.02256	-2.95735	-2.89101	-2.82359
0.9800	-2.66657	-2.62631	-2.58480	-2.54206	-2.49811	-2.45298	-2.40670
0.9750	-2.48855	-2.45482	-2.41984	-2.38364	-2.34623	-2.30764	-2.26790
0.9600	-2.10823	-2.08758	-2.06573	-2.04269	-2.01848	-1.99311	-1.96660
0.9500	-1.92472	-1.90992	-1.89395	-1.87683	-1.85856	-1.83916	-1.81864
0.9000	-1.33904	-1.34047	-1.34092	-1.34039	-1.33889	-1.33640	-1.33294
0.8000	-0.71915	-0.73257	-0.74537	-0.75752	-0.76902	-0.77986	-0.79002
0.7000	-0.33054	-0.34772	-0.36458	-0.38111	-0.39729	-0.41309	-0.42851
0.6000	-0.03560	-0.05297	-0.07032	-0.08763	-0.10486	-0.12199	-0.13901
0.5704	0.04116	0.02421	0.00719	-0.00987	-0.02693	-0.04397	-0.06097
0.5000	0.21040	0.19517	0.17968	0.16397	0.14807	0.13199	0.11578
0.4296	0.36620	0.35370	0.34075	0.32740	0.31368	0.29961	0.28516
0.4000	0.42899	0.41794	0.40638	0.39434	0.38186	0.36889	0.35565
0.3000	0.63400	0.62944	0.62415	0.61815	0.61146	0.60412	0.59615
0.2000	0.83841	0.84369	0.84809	0.85161	0.85426	0.85607	0.85703
0.1000	1.06413	1.08608	1.10726	1.12762	1.14712	1.16574	1.18347
0.0500	1.20578	1.24313	1.28019	1.31684	1.35299	1.38855	1.42345
0.0400	1.24028	1.28225	1.32414	1.36584	1.40720	1.44813	1.48852
0.0250	1.30042	1.35153	1.40314	1.45507	1.50712	1.55914	1.61099
0.0200	1.32412	1.37929	1.43529	1.49188	1.54886	1.60604	1.66325
0.0100	1.38267	1.44942	1.51808	1.58838	1.66001	1.73271	1.80621
0.0050	1.42439	1.50114	1.58110	1.66390	1.74919	1.83660	1.92580
0.0020	1.46232	1.55016	1.64305	1.74062	1.84244	1.94806	2.05701
0.0010	1.48216	1.57695	1.67825	1.78572	1.89894	2.01739	2.14053
0.0005	1.49673	1.59738	1.70603	1.82241	1.94611	2.07661	2.21328
0.0001	1.51752	1.62838	1.75053	1.88410	2.02891	2.18448	2.35015

Table 4.13. Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

	Skew						
Prob.	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
0.9999	-5.04718	-4.82141	-4.59687	-4.37394	-4.15301	-3.93453	-3.71902
0.9995	-4.29311	-4.12443	-3.95605	-3.78820	-3.62113	-3.45513	-3.29053
0.9990	-3.95567	-3.81090	-3.66608	-3.52139	-3.37703	-3.23322	-3.09023
0.9980	-3.60872	-3.48737	-3.36566	-3.24371	-3.12169	-2.99978	-2.87816
0.9950	-3.13232	-3.04102	-2.94900	-2.85636	-2.76321	-2.66965	-2.57583
0.9900	-2.75514	-2.68572	-2.61539	-2.54421	-2.47226	-2.39961	-2.32635
0.9800	-2.35931	-2.31084	-2.26133	-2.21081	-2.15935	-2.10697	-2.05375
0.9750	-2.22702	-2.18505	-2.14202	-2.09795	-2.05290	-2.00688	-1.95996
0.9600	-1.93896	-1.91022	-1.88039	-1.84949	-1.81756	-1.78462	-1.75069
0.9500	-1.79701	-1.77428	-1.75048	-1.72562	-1.69971	-1.67279	-1.64485
0.9000	-1.32850	-1.32309	-1.31671	-1.30936	-1.30105	-1.29178	-1.28155
0.8000	-0.79950	-0.80829	-0.81638	-0.82377	-0.83044	-0.83639	-0.84162
0.7000	-0.44352	-0.45812	-0.47228	-0.48600	-0.49927	-0.51207	-0.52440
0.6000	-0.15589	-0.17261	-0.18916	-0.20552	-0.22168	-0.23763	-0.25335
0.5704	-0.07791	-0.09178	-0.11154	-0.12820	-0.14472	-0.16111	-0.17733
0.5000	0.09945	0.08302	0.06651	0.04993	0.03325	0.01662	0.00000
0.4296	0.27047	0.25558	0.24037	0.22492	0.20925	0.19339	0.17733
0.4000	0.34198	0.32796	0.31362	0.29897	0.28403	0.26882	0.25335
0.3000	0.58757	0.57840	0.56867	0.55839	0.54757	0.53624	0.52440
0.2000	0.85718	0.85653	0.85508	0.85285	0.84986	0.84611	0.84162
0.1000	1.20028	1.21618	1.23114	1.24516	1.25824	1.27037	1.28155
0.0500	1.45762	1.49101	1.52357	1.55527	1.58607	1.61594	1.64485
0.0400	1.52830	1.56740	1.60574	1.64329	1.67999	1.71580	1.75069
0.0250	1.66253	1.71366	1.76427	1.81427	1.86360	1.91219	1.95996
0.0200	1.72033	1.77716	1.83361	1.88959	1.94499	1.99973	2.05375
0.0100	1.88029	1.95472	2.02933	2.10394	2.17840	2.25258	2.32635
0.0050	2.01644	2.10825	2.20092	2.29423	2.38795	2.48187	2.57583
0.0020	2.16884	2.28311	2.39942	2.51741	2.63672	2.75706	2.87816
0.0010	2.26780	2.39867	2.53261	2.66915	2.80786	2.94834	3.09023
0.0005	2.35549	2.50257	2.65390	2.80889	2.96698	3.12767	3.29053
0.0001	2.52507	2.70836	2.89907	3.09631	3.29921	3.50703	3.71902

Table 4.13. Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

Prob.	Skew						
	0.1	0.2	0.3	0.4	0.5	0.6	0.7
0.9999	-3.50703	-3.29921	-3.09631	-2.89907	-2.70836	-2.52507	-2.35015
0.9995	-3.12767	-2.96698	-2.80889	-2.65390	-2.50257	-2.35549	-2.21328
0.9990	-2.94834	-2.80786	-2.66915	-2.53261	-2.39867	-2.26780	-2.14053
0.9980	-2.75706	-2.63672	-2.51741	-2.39942	-2.28311	-2.16884	-2.05701
0.9950	-2.48187	-2.38795	-2.29423	-2.20092	-2.10825	-2.01644	-1.92580
0.9900	-2.25258	-2.17840	-2.10394	-2.02933	-1.95472	-1.88029	-1.80621
0.9800	-1.99973	-1.94499	-1.88959	-1.83361	-1.77716	-1.72033	-1.66325
0.9750	-1.91219	-1.86360	-1.81427	-1.76427	-1.71366	-1.66253	-1.61099
0.9600	-1.71580	-1.67999	-1.64329	-1.60574	-1.56740	-1.52830	-1.48852
0.9500	-1.61594	-1.58607	-1.55527	-1.52357	-1.49101	-1.45762	-1.42345
0.9000	-1.27037	-1.25824	-1.24516	-1.23114	-1.21618	-1.20028	-1.18347
0.8000	-0.84611	-0.84986	-0.85285	-0.85508	-0.85653	-0.85718	-0.85703
0.7000	-0.53624	-0.54757	-0.55839	-0.56867	-0.57840	-0.58757	-0.59615
0.6000	-0.26882	-0.28403	-0.29897	-0.31362	-0.32796	-0.34198	-0.35565
0.5704	-0.19339	-0.20925	-0.22492	-0.24037	-0.25558	-0.27047	-0.28516
0.5000	-0.01662	-0.03325	-0.04993	-0.06651	-0.08302	-0.09945	-0.11578
0.4296	0.16111	0.14472	0.12820	0.11154	0.09478	0.07791	0.06097
0.4000	0.23763	0.22168	0.20552	0.18916	0.17261	0.15589	0.13901
0.3000	0.51207	0.49927	0.48600	0.47228	0.45812	0.44352	0.42851
0.2000	0.83639	0.83044	0.82377	0.81638	0.80829	0.79950	0.79002
0.1000	1.29178	1.30105	1.30936	1.31671	1.32309	1.32850	1.33294
0.0500	1.67279	1.69971	1.72562	1.75048	1.77428	1.79701	1.81864
0.0400	1.78462	1.81756	1.84949	1.88039	1.91022	1.93896	1.96660
0.0250	2.00688	2.05290	2.09795	2.14202	2.18505	2.22702	2.26790
0.0200	2.10697	2.15935	2.21081	2.26133	2.31084	2.35931	2.40670
0.0100	2.39961	2.47226	2.54421	2.61539	2.68572	2.75514	2.82359
0.0050	2.66965	2.76321	2.85636	2.94900	3.04102	3.13232	3.22281
0.0020	2.99978	3.12169	3.24371	3.36566	3.48737	3.60872	3.72957
0.0010	3.23322	3.37703	3.52139	3.66608	3.81090	3.95567	4.10022
0.0005	3.45513	3.62113	3.78820	3.95605	4.12443	4.29311	4.46189
0.0001	3.93453	4.15301	4.37394	4.59687	4.82141	5.04718	5.27389

Table 4.13. Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

	Skew						
Prob.	**0.8**	**0.9**	**1.0**	**1.1**	**1.2**	**1.3**	**1.4**
0.9999	2.18448	-2.02891	-1.88410	-1.75053	-1.62838	-1.51752	-1.41753
0.9995	-2.07661	-1.94611	-1.82241	-1.70603	-1.59738	-1.49673	-1.40413
0.9990	-2.01739	-1.89894	-1.78572	-1.67825	-1.57695	-1.48216	-1.39408
0.9980	-1.94806	-1.84244	-1.74062	-1.64305	-1.55016	-1.46232	-1.37981
0.9950	-1.83660	-1.74919	-1.66390	-1.58110	-1.50114	-1.42439	-1.35114
0.9900	-1.73271	-1.66001	-1.58838	-1.51808	-1.44942	-1.38267	-1.31815
0.9800	-1.60604	-1.54886	-1.49188	-1.43529	-1.37929	-1.32412	-1.26999
0.9750	-1.55914	-1.50712	-1.45507	-1.40314	-1.35153	-1.30042	-1.25004
0.9600	-1.44813	-1.40720	-1.36584	-1.32414	-1.28225	-1.24028	-1.19842
0.9500	-1.38855	-1.35299	-1.31684	-1.28019	-1.24313	-1.20578	-1.16827
0.9000	-1.16574	-1.14712	-1.12762	-1.10726	-1.08608	-1.06413	-1.04144
0.8000	-0.85607	-0.85426	-0.85161	-0.84809	-0.84369	-0.83841	-0.83223
0.7000	-0.60412	-0.61146	-0.61815	-0.62415	-0.62944	-0.63400	-0.63779
0.6000	-0.36889	-0.38186	-0.39434	-0.40638	-0.41794	-0.42899	-0.43949
0.5704	-0.29961	-0.31368	-0.32740	-0.34075	-0.35370	-0.36620	-0.37824
0.5000	-0.13199	-0.14807	-0.16397	-0.17968	-0.19517	-0.21040	-0.22535
0.4296	0.04397	0.02693	0.00987	-0.00719	-0.02421	-0.04116	-0.05803
0.4000	0.12199	0.10486	0.08763	0.07032	0.05297	0.03560	0.01824
0.3000	0.41309	0.39729	0.38111	0.36458	0.34772	0.33054	0.31307
0.2000	0.77986	0.76902	0.75752	0.74537	0.73257	0.71915	0.70512
0.1000	1.33640	1.33889	1.34039	1.34092	1.34047	1.33904	1.33665
0.0500	1.83916	1.85856	1.87683	1.89395	1.90992	1.92472	1.93836
0.0400	1.99311	2.01848	2.04269	2.06573	2.08758	2.10823	2.12768
0.0250	2.30764	2.34623	2.38364	2.41984	2.45482	2.48855	2.52102
0.0200	2.45298	2.49811	2.54206	2.58480	2.62631	2.66657	2.70556
0.0100	2.89101	2.95735	3.02256	3.08660	3.14944	3.21103	3.27134
0.0050	3.31243	3.40109	3.48874	3.57530	3.66073	3.74497	3.82798
0.0020	3.84981	3.96932	4.08802	4.20582	4.32263	4.43839	4.55304
0.0010	4.24439	4.38807	4.53112	4.67344	4.81492	4.95549	5.09505
0.0005	4.63057	4.79899	4.96701	5.13449	5.30130	5.46735	5.63252
0.0001	5.50124	5.72899	5.95691	6.18480	6.41249	6.63980	6.86661

Table 4.13. Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

Prob.	Skew					
	1.5	1.6	1.7	1.8	1.9	2.0
0.9999	-1.32774	-1.24728	-1.17520	-1.11054	-1.05239	-0.99990
0.9995	-1.31944	-1.24235	-1.17240	-1.10901	-1.05159	-0.99950
0.9990	-1.31275	-1.23805	-1.16974	-1.10743	-1.50568	-0.99900
0.9980	-1.30279	-1.23132	-1.16534	-1.10465	-1.04898	-0.99800
0.9950	-1.28167	-1.21618	-1.15477	-1.09749	-1.04427	-0.99499
0.9900	-1.25611	-1.19680	-1.14042	-1.08711	-1.03695	-0.98995
0.9800	-1.21716	-1.16584	-1.11628	-1.06864	-1.02311	-0.97980
0.9750	-1.20059	-1.15229	-1.10537	-1.06001	-1.01640	-0.97468
0.9600	-1.15682	-1.11566	-1.07513	-1.03543	-0.99672	-0.95918
0.9500	-1.13075	-1.09338	-1.05631	-1.01973	-0.98381	-0.94871
0.9000	-1.01810	-0.99418	-0.96977	-0.94496	-0.91988	-0.89464
0.8000	-0.82516	-0.81720	-0.80837	-0.79868	-0.78816	-0.77686
0.7000	-0.64080	-0.64300	-0.64436	-0.64488	-0.64453	-0.64333
0.6000	-0.44942	-0.45873	-0.46739	-0.47538	-0.48265	-0.48917
0.5704	-0.38977	-0.40075	-0.41116	-0.42095	-0.43008	-0.43854
0.5000	-0.23996	-0.25422	-0.26808	-0.28150	-0.29443	-0.30685
0.4296	-0.07476	-0.09132	-0.10769	-0.12381	-0.13964	-0.15516
0.4000	0.00092	-0.01631	-0.03344	-0.05040	-0.06718	-0.08371
0.3000	0.29535	0.27740	0.25925	0.24094	0.22250	0.20397
0.2000	0.69050	0.67532	0.65959	0.64335	0.62662	0.60944
0.1000	1.33330	1.32900	1.32376	1.31760	1.31054	1.30259
0.0500	1.95083	1.96213	1.97227	1.98124	1.98906	1.99573
0.0400	2.14591	2.16293	2.17873	2.19332	2.20670	2.21888
0.0250	2.55222	2.58214	2.61076	2.63810	2.66413	2.68888
0.0200	2.74325	2.77964	2.81472	2.84848	2.88091	2.91202
0.0100	3.33035	3.38804	3.44438	3.49935	3.55295	3.60517
0.0050	3.90973	3.99016	4.06926	4.14700	4.22336	4.29832
0.0020	4.66651	4.77875	4.88971	4.99937	5.10768	5.21461
0.0010	5.23353	5.37087	5.50701	5.64190	5.77549	5.90776
0.0005	5.79673	5.95990	6.12196	6.28285	6.44251	6.60090
0.0001	7.09277	7.31818	7.54272	7.76632	7.98888	8.21034

Table 4.14. Calculation of Log-Pearson Type III Discharges for Medina River Using Station Skew

(1) Return Period (yrs)	(2) Exceedence Probability	(3) K	SI Unit (4) Y	SI Unit (5) X (m^3/s)	CU Unit (6) Y	CU Unit (7) X (ft^3/s)
2	0.50	-0.03325	2.078	120	3.626	4,230
5	0.20	0.83044	2.418	262	3.966	9,250
10	0.10	1.30105	2.604	402	4.152	14,200
25	0.04	1.81756	2.807	641	4.355	22,600
50	0.02	2.15935	2.942	875	4.490	30,900
100	0.01	2.47226	3.065	1,160	4.613	41,000

(3) from Table 4.13 for G = 0.2 (rounded from 0.236)

(4) $Y = \overline{Y} + KS_y = 2.091 + 0.394K$

(5) $X = 10^Y$

(6) $Y = \overline{Y} + KS_y = 3.639 + 0.394K$

(7) $X = 10^Y$

Table 4.15. Calculation of Log-Pearson Type III Discharges for Medina River Using Generalized Skew

(1) Return Period (yrs)	(2) Exceedence Probability	(3) K	SI Unit (4) Y	SI Unit (5) X (m^3/s)	CU Unit (6) Y	CU Unit (7) X (ft^3/s)
2	0.50	0.04993	2.111	129	3.659	4,560
5	0.20	0.85285	2.427	267	3.975	9,440
10	0.10	1.24516	2.582	382	4.130	13,500
25	0.04	1.64329	2.738	547	4.286	19,300
50	0.02	1.88959	2.836	685	4.383	24,200
100	0.01	2.10394	2.920	832	4.468	29,400

(3) from Table 4.13 for $\overline{G}$ = -0.3 (rounded from -0.252)

(4) $Y = \overline{Y} + KS_y = 2.091 + 0.394K$

(5) $X = 10^Y$

(6) $Y = \overline{Y} + KS_y = 3.639 + 0.394K$

(7) $X = 10^Y$

Table 4.16. Calculation of Log-Pearson Type III Discharges for Medina River Using Weighted Skew

(1) Return Period (yrs)	(2) Exceedence probability	(3) K	SI Unit (4) Y	SI Unit (5) X(m³/s)	CU Unit (6) Y	CU Unit (7) X (ft³/s)
2	0.50	-0.01662	2.085	121	3.632	4,290
5	0.20	0.83639	2.421	264	3.969	9,310
10	0.10	1.29178	2.600	398	4.148	14,100
25	0.04	1.78462	2.794	622	4.342	22,000
50	0.02	2.10697	2.922	836	4.469	29,400
100	0.01	2.39961	3.036	1,090	4.584	38,400

(3) from Table 4.13 for G_W = 0.1 (rounded from 0.084)

(4) $Y = \overline{Y} + KS_y = 2.091 + 0.394K$

(5) $X = 10^Y$

(6) $Y = \overline{Y} + KS_y = 3.639 + 0.394K$

(7) $X = 10^Y$

4.3.5 Evaluation of Flood Frequency Predictions

The peak flow data for the Medina River gage have now been analyzed by four different frequency distributions and, in the case of log-Pearson Type III distribution, by three different options of skew. The two-parameter log-normal distribution is a special case of the log-Pearson Type III distribution, specifically when the skew is zero. The normal and Gumbel distributions assume fixed skews of zero and 1.139, respectively, for the untransformed data.

The log-Pearson Type III distribution, which uses three parameters, should be superior to all three of the two-parameter distributions discussed in this document. The predicted 10-year and 100-year floods obtained by each of these methods are summarized in Table 4.17. There is considerable variation in the estimates, especially for the 100-year flood, where the values range from 653 m^3/s (23,100 ft^3/s) to 1160 m^3/s (41,000 ft^3/s).

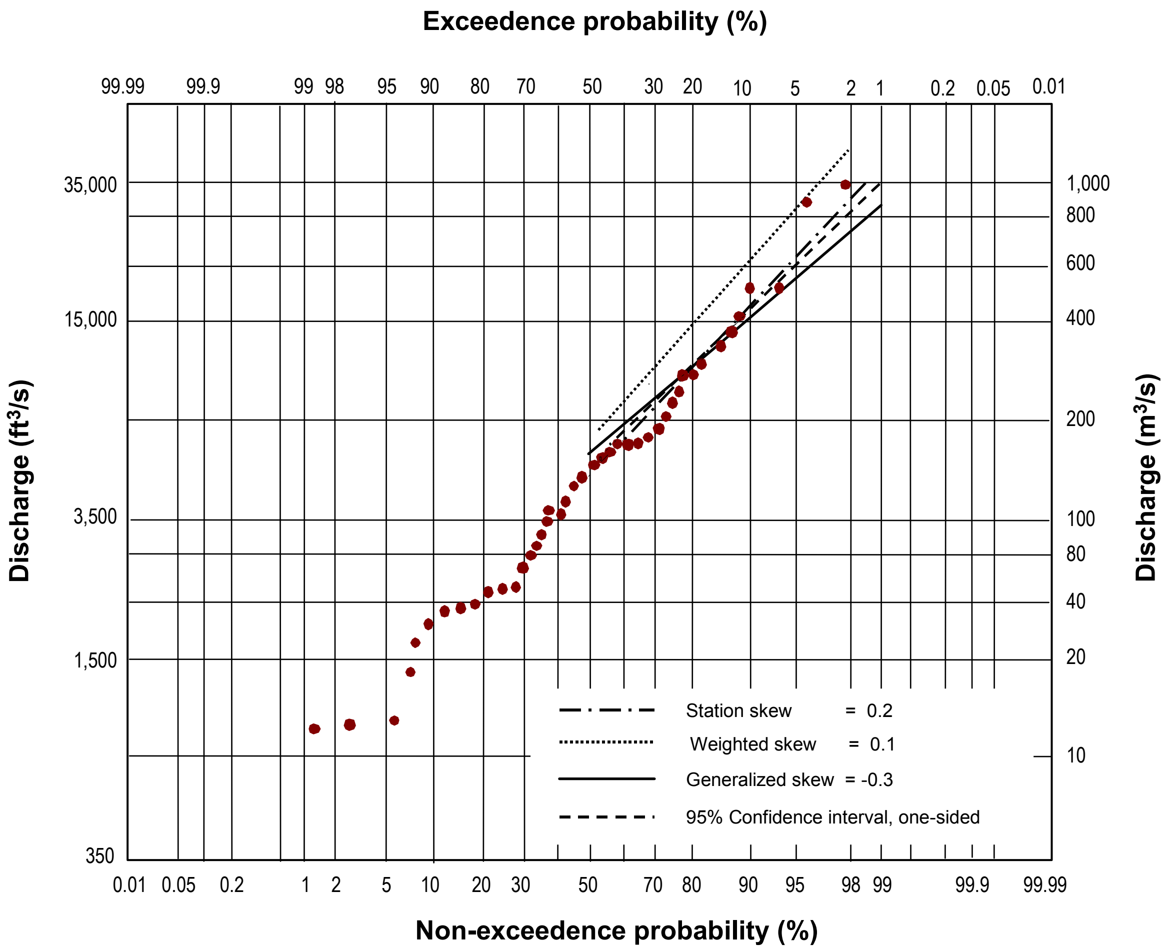


Figure 4.15. Log-Pearson Type III distribution frequency curve, Medina River

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 scales. Based on this preliminary graphical analysis, as well as judgment, some standard distributions might be eliminated before the frequency analysis is begun.

Frequently, more than one distribution or, in the case of the log-Pearson Type 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, both of which are discussed below.

Table 4.17. Summary of 10- and 100-year Discharges for Selected Probability Distributions

Distribution	Estimated Flow			
	SI (m^3/s)		Customary (ft^3/s)	
	10-yr	100-yr	10-yr	100-yr
Normal	444	653	15,700	23,100
Log-normal	394	1,020	13,900	35,900
Gumbel	485	895	17,100	31,600
Log-Pearson Type III				
Station Skew (G = 0.2)	402	1,160	14,200	41,000
Generalized Skew ($\bar{G}$ = -0.3)	382	832	13,500	29,400
Weighted Skew (G_W = 0.1)	398	1,090	14,100	38,400

4.3.5.1 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 for the mean, standard deviation, and coefficient of skew as:

$$\text{Mean}: \quad S_{TM} = \frac{S}{n^{0.5}} \tag{4.37}$$

$$\text{Standard Deviation}: \quad S_{TS} = \frac{S}{(2n)^{0.5}} \tag{4.38}$$

$$\text{Coefficient of Skew}: \quad S_{TG} = \left[\frac{6n(n-1)}{(n-2)(n+1)(n+3)}\right]^{0.5} \tag{4.39}$$

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.

The standard error of estimate 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 a reflection of a short record.

Using the Medina River annual flood series as an example, the standard errors for the parameters of the log-Pearson Type III computed from Equations 4.37, 4.38, and 4.39 for the logarithms are:

$$S_{TM} = 0.394/(43)^{0.5} = 0.060$$

$$S_{TS} = 0.394/(2(43))^{0.5} = 0.0425$$

$$S_{TG} = [6(43)(42)/((41)(44)(46))]^{0.5} = 0.361$$

The standard error for the skew coefficient of 0.361 is relatively large. 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 reflected in the standard error of estimate of the skew coefficient. For this reason, some hydrologists prefer confidence limits for evaluating the reliability of a selected frequency distribution.

4.3.5.2 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 a sample estimate of a population, it is probable that another sample taken at the same location and 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 a specified confidence level.

USGS Bulletin 17B (1982) outlines a method for developing upper and lower confidence intervals. The general forms of the confidence limits are:

$$U_{p,c}(Q) = \overline{Q} + S\,K^{U}_{p,c} \qquad (4.40)$$

and

$$L_{p,c}(Q) = \overline{Q} + S\,K^{L}_{p,c} \qquad (4.41)$$

where,

- c = level of confidence
- p = exceedence probability
- $U_{p,c}(Q)$ = upper confidence limit corresponding to the values of p and c, for flow Q
- $L_{p,c}(Q)$ = lower confidence limit corresponding to the values of p and c, for flow Q
- $K^{U}_{p,c}$ = upper confidence coefficient corresponding to the values of p and c
- $K^{L}_{p,c}$ = lower confidence coefficient corresponding to the values of p and c

Values of $K^{U}_{p,c}$ and $K^{L}_{p,c}$ for the normal distribution are given in Table 4.18 for the commonly used confidence levels of 0.05 and 0.95. USGS Bulletin 17B (1982), from which Table 4.18 was abstracted, contains a more extensive table covering other confidence levels.

Confidence limits defined in this manner and with the values of Table 4.18 are called one-sided because each defines the limit on just one side of the frequency curve; for 95 percent confidence only one of the values should be computed. The one-sided limits can be combined to form a two-sided confidence interval such that the combination of 95 percent and 5 percent confidence limits define a two-sided 90 percent confidence interval. Practically, this means that at a specified exceedence probability or return period, there is a 5 percent chance the flow will exceed the upper confidence limit and a 5 percent chance the flow will be less than the lower confidence limit. Stated another way, it can be expected that, 90 percent of the time, the specified frequency flow will fall within the two confidence limits.

Table 4.18. Confidence Limit Deviate Values for Normal and Log-normal Distributions (from USGS Bulletin 17B, 1982)

Confidence Level	Systematic Record n	Exceedence Probability								
		0.002	0.010	0.020	0.040	0.100	0.200	0.500	0.800	0.990
0.05	10	4.862	3.981	3.549	3.075	2.355	1.702	0.580	-0.317	-1.563
	15	4.304	3.520	3.136	2.713	2.068	1.482	0.455	-0.406	-1.677
	20	4.033	3.295	2.934	2.534	1.926	1.370	0.387	-0.460	-1.749
	25	3.868	3.158	2.809	2.425	1.838	1.301	0.342	-0.497	-1.801
	30	3.755	3.064	2.724	2.350	1.777	1.252	0.310	-0.525	-1.840
	40	3.608	2.941	2.613	2.251	1.697	1.188	0.266	-0.556	-1.896
	50	3.515	2.862	2.542	2.188	1.646	1.146	0.237	-0.592	-1.936
	60	3.448	2.807	2.492	2.143	1.609	1.116	0.216	-0.612	-1.966
	70	3.399	2.765	2.454	2.110	1.581	1.093	0.199	-0.629	-1.990
	80	3.360	2.733	2.425	2.083	1.559	1.076	0.186	-0.642	-2.010
	90	3.328	2.706	2.400	2.062	1.542	1.061	0.175	-0.652	-2.026
	100	3.301	2.684	2.380	2.044	1.527	1.049	0.166	-0.662	-2.040
0.95	10	1.989	1.563	1.348	1.104	0.712	0.317	-0.580	-1.702	-3.981
	15	2.121	1.677	1.454	1.203	0.802	0.406	-0.455	-1.482	-3.520
	20	2.204	1.749	1.522	1.266	0.858	0.460	-0.387	-1.370	-3.295
	25	2.264	1.801	1.569	1.309	0.898	0.497	-0.342	-1.301	-3.158
	30	2.310	1.840	1.605	1.342	0.928	0.525	-0.310	-1.252	-3.064
	40	2.375	1.896	1.657	1.391	0.970	0.565	-0.266	-1.188	-2.941
	50	2.421	1.936	1.694	1.424	1.000	0.592	-0.237	-1.146	-2.862
	60	2.456	1.966	1.722	1.450	1.022	0.612	-0.216	-1.116	-2.807
	70	2.484	1.990	1.745	1.470	1.040	0.629	-0.199	-1.093	-2.765
	80	2.507	2.010	1.762	1.487	1.054	0.642	-0.186	-1.076	-2.733
	90	2.526	2.026	1.778	1.500	1.066	0.652	-0.175	-1.061	-2.706
	100	2.542	2.040	1.791	1.512	1.077	0.662	-0.166	-1.049	-2.684

When the skew is non-zero, USGS Bulletin 17B (1982) 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 exceedence probability:

$$K^{U}_{P,C} = \frac{K_{G,P} + \left(K^{2}_{G,P} - a\,b\right)^{0.5}}{a} \quad (4.42a)$$

and

$$K^{L}_{P,C} = \frac{K_{G,P} - \left(K^{2}_{G,P} - a\,b\right)^{0.5}}{a} \quad (4.42b)$$

where

$$a = 1 - \frac{Z^{2}_{c}}{2\,(n - 1)} \quad (4.42c)$$

$$b = K^{2}_{G,P} - \frac{Z^{2}_{c}}{n} \quad (4.42d)$$

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

Confidence intervals were computed for the Medina River flood series using the USGS Bulletin 17B (1982) procedures for both the log-normal and the log-Pearson Type III distributions. The weighted skew of 0.1 was used with the log-Pearson Type III analysis. The computations for the confidence intervals are given in Tables 4.19 (log-normal) and 4.20 (log-Pearson Type III). The confidence intervals for the log-normal and log-Pearson Type III are shown in Figures 4.13 and 4.15, respectively.

It appears that a log-Pearson Type 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. Based on this analysis, the log-Pearson Type III would be the preferred standard distribution with the log-normal also acceptable. The normal and Gumbel distributions are unsatisfactory for this particular set of data.

Table 4.19. Computation of One-sided, 95 Percent Confidence Interval for the Log-normal Analysis of the Medina River Annual Maximum Series

			SI			CU		
(1) Return Period (yrs)	(2) Exceedence Probability	(3) K^u	(4) U	(5) X^u (m^3/s)	(6) X (m^3/s)	(7) U	(8) X^u (ft^3/s)	(9) X (ft^3/s)
2	0.5	0.2573	2.192	156	123	3.740	5,500	4,360
5	0.2	1.1754	2.554	358	265	4.102	12,600	9,350
10	0.1	1.6817	2.754	568	394	4.302	20,000	13,900
25	0.04	2.2321	2.970	935	604	4.518	33,000	21,300
50	0.02	2.5917	3.112	1,300	795	4.660	45,700	28,100
100	0.01	2.9173	3.241	1,740	1,020	4.788	61,400	35,900
500	0.002	3.5801	3.502	3,180	1,680	5.050	112,200	59,300

(3) interpolated from Table 4.18 for a record length of 43 years

(4) $U = \overline{Y} + S_y K^U = 2.091 + 0.394\,K^U$

(5) $X^U = 10^U$

(6) estimated using Equations 4.29 and 4.30

(7) $U = \overline{Y} + S_y K^U = 3.639 + 0.394\ K^U$
(8) $X^U = 10^U$
(9) estimated using Equations 4.29 and 4.30

Table 4.20. Computation of One-sided, 95 Percent Confidence Interval for the Log-Pearson Type III Analysis of the Medina River Annual Maximum Series with Weighted Skew

					SI			CU		
(1) Return Period (yrs)	(2) Exceedence Probability	(3) K	(4) b	(5) K^U	(6) U	(7) X^U (m^3/s)	(8) X (m^3/s)	(9) U	(10) X^U (ft^3/s)	(11) X (ft^3/s)
2	0.5	-0.01662	-0.0627	0.2378	2.185	153	121	3.733	5,410	4,290
5	0.2	0.83639	0.6366	1.1627	2.549	354	264	4.097	12,500	9,310
10	0.1	1.29178	1.6058	1.6847	2.755	569	398	4.303	20,090	14,060
25	0.04	1.78462	3.1219	2.2618	2.982	959	622	4.530	33,880	21,980
50	0.02	2.10697	4.3764	2.6437	3.133	1,360	834	4.681	47,970	29,440
100	0.01	2.39961	5.6952	2.9924	3.270	1,860	1,090	4.818	65,770	38,370
500	0.002	2.99978	8.9357	3.7116	3.553	3,570	1,870	5.101	126,180	66,220

(3) from Table 4.13 for skew *G* = 0.1

(4) from Equation 4.42d

$$b = K^2 - \frac{Z_c^2}{n} = K^2 - \frac{(1.645)^2}{43} = K^2 - 0.06293$$

(5) from Equation 4.42a

$$K^U = \frac{K + (K^2 - ab)^{0.5}}{a} = \frac{K + (K^2 - 0.96779\ b)^{0.5}}{0.96779}$$

(6) from Equation 4.40

$$U = \overline{Y} + S_Y K^U = 2.091 + 0.394 K^U$$

(7) from Equation 4.35

$$X^U = 10^U$$

(8) from Table 4.16

(9) from Equation 4.40

$$U = \overline{Y} + S_Y K^U = 3.639 + 0.394 K^U$$

(10) from Equation 4.35

$$X^U = 10^U$$

(11) from Table 4.16

4.3.6 Other 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.

4.3.6.1 Outliers

Outliers, which may be found at either or both ends of a frequency distribution, are measured values that occur, but appear to be from a longer sample or different population. This is reflected when one or more data points do not follow the trend of the remaining data.

USGS Bulletin 17B (1982) 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 ± 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 following equations, respectively:

$$Y_H = \overline{Y} + K_N S_Y \tag{4.43}$$

and

$$Y_L = \overline{Y} - K_N S_Y \tag{4.44}$$

where,

Y_H, Y_L = log of the high or low outlier limit, respectively
$\overline{Y}$ = mean of the log of the sample flows
S_y = standard deviation of the sample
K_N = critical deviate (from Table 4.21).

If the sample is found to contain high outliers, the peak flows should be checked against other historical data sources and data from nearby stations. This check enables categorization of the flow observation as a potential anomaly or error in the sample. USGS Bulletin 17B (1982) recommends that high outliers be adjusted for historical information or retained in the sample as a systematic peak. The high outlier should not be discarded unless the peak flow is shown to be seriously in error. 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.

To test for low outliers, the low outlier threshold Y_L of Equation 4.44 is computed. The corresponding discharge $X_L = 10^{Y_L}$ is then computed. If any discharges in the flood series are less than X_L, then they are considered to be low outliers and should be deleted from the sample. The moments should be recomputed and the conditional probability adjustment from the arid lands hydrology section of Chapter 9 (Special Topics) applied.

Table 4.21. Outlier Test Deviates (K_N) at 10 Percent Significance Level (from USGS Bulletin 17B, 1982)

Sample Size	K_N Value	Sample Size	K_N Value	Sample Size	K_N Value	Sample Size	K_N 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	2.989	125	3.092
21	2.408	56	2.811	91	2.984	126	3.095
22	2.429	57	2.818	92	2.889	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	66	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	2.720	79	2.935	114	3.061	149	3.148

Example 4.10. To illustrate these criteria for outlier detection, Equations 4.43 and 4.44 are applied to the 43-year record for the Medina River, which has a log mean of 2.091 (3.639 in CU units) and a log standard deviation of 0.394. From Table 4.21, K_N = 2.710.

Testing first for high outliers:

Variable	**Value in SI**	**Value in CU**
Y_H	$2.091 + 2.710\ (0.394) = 3.159$	$3.639 + 2.710\ (0.394) = 4.707$
X_H	$10^{3.159} = 1{,}440\ m^3/s$	$10^{4.707} = 50{,}900\ ft^3/s$

No flows in the sample exceed this amount, so there are no high outliers. Testing for low outliers, Equation 4.44 gives:

Variable	**Value in SI**	**Value in CU**
Y_L	$2.091 - 2.710\ (0.394) = 1.023$	$3.639 - 2.710\ (0.394) = 2.571$
X_L	$10^{1.023} = 11\ m^3/s$	$10^{2.571} = 372\ ft^3/s$

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

4.3.6.2 Historical Data

When reliable information indicates 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 random 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 adjusted to reflect current watershed conditions.

USGS Bulletin 17B (1982) 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 weight of 1.00 to the historic event, and weighting the order of the station data by a value determined from the equation:

$$W = \frac{H - Z}{n + L} \qquad (4.45)$$

where,

W = the weighting factor

H = the length of the historic period of years
Z = the number of historical events included in the analysis
L = the number of low outliers excluded from the analysis.

The properties of the historically extended sample are then computed according to the equations

$$\overline{Q}_L{}' = \frac{W \sum Q_L + \sum Q_{L,Z}}{H - WL} \qquad (4.46)$$

$$(S_L{}')^2 = \frac{W \sum (Q_L - Q_L{}')^2 + \sum (Q_{L,Z} - \overline{Q}_L{}')^2}{H - WL - 1} \qquad (4.47)$$

and

$$G_L{}' = \frac{H - W\,L}{(H - W\,L - 1)(H - WL - 2)} \left[\frac{W \sum (Q_L - \overline{Q}_L{}')^3 + \sum (Q_{L,Z} - \overline{Q}_L{}')^3}{(S_L{}')^3} \right] \qquad (4.48)$$

where,

$\overline{Q}_L{}'$ = historically adjusted mean log transform of the flows
Q_L = log transform of the flows contained in the sample record
$Q_{L,Z}$ = log of the historic peak flow
$S_L{}'$ = historically adjusted standard deviation
$G_L{}'$ = 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, USGS Bulletin 17B (1982) gives the following adjustments that can be applied directly

$$\overline{Q}_L{}' = \frac{W\,n\,\overline{Q}_L + \sum Q_{L,Z}}{H - W\,L} \qquad (4.49)$$

$$(S_L{}')^2 = \frac{W(n-1)\,S_L^2 + W\,n(\overline{Q}_L - \overline{Q}_L{}')^2 + \sum (Q_{L,Z} - \overline{Q}_L{}')^2}{H - W\,L - 1} \qquad (4.50)$$

$$G_L{}' = \frac{H - WL}{(H - WL - 1)(H - WL - 2)(S_L{}')^3} \times \left[\frac{W(n-1)(n-2)\,S_L^3 G_L}{n} + 3W(n-1)(\overline{Q}_L - \overline{Q}_L{}')\,S_L^2 + Wn(\overline{Q}_L - \overline{Q}_L{}')^3 + \sum (Q_{L,Z} - \overline{Q}_L{}')^3 \right] \qquad (4.51)$$

Once the adjusted statistical parameters are determined, the log-Pearson Type III distribution is determined by Equation 4.27 using the Weibull plotting position formula:

$$P = \frac{m'}{H + 1} \qquad (4.52)$$

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

$$m' = m \qquad \text{for } 1 \leq m \leq Z$$

$$m' = Wm - (W - 1)(Z + 0.5) \qquad \text{for } (Z + 1) \leq m \leq (Z + nL)$$

Detailed examples illustrating the computations for the historic adjustment are contained in USGS Bulletin 17B (1982) and the designer should consult this reference for further information.

4.3.6.3 Incomplete Records and Zero Flows

Stream flow 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 that 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 further adjustment.

Zero flows or flows that are too low to be recorded present more of a problem because, in the log transform, these flows produce undefined values. In this case, USGS Bulletin 17B (1982) presents an adjustment based on conditional probability that 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, P_a, that the annual peak will exceed the level below where either flows are zero or not considered (the truncation level):

$$P_a = \frac{M}{n} \qquad (4.53)$$

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

$$P = P_a \, P_d \qquad (4.54)$$

where P_d is the selected probability.

Since the frequency curve adjusted by Equation 4.54 has unknown statistics, its properties, synthetic values, are computed by the equations:

$$\overline{Q}_s = log(Q_{0.50}) - K_{0.50}(S_S) \quad (4.55)$$

$$S_S = \frac{log(Q_{0.01} / Q_{0.50})}{K_{0.01} - K_{0.50}} \quad (4.56)$$

and

$$G_S = -2.50 + 3.12 \left[\frac{log(Q_{0.01} / Q_{0.10})}{log(Q_{0.10} / Q_{0.50})} \right] \quad (4.57)$$

where $\overline{Q}_s$, S_s, and G_s are the mean, standard deviation, and skew of the synthetic frequency curve, $Q_{0.01}$, $Q_{0.10}$, and $Q_{0.50}$ are discharges with exceedence probabilities of 0.01, 0.10 and 0.50, respectively, and $K_{0.01}$ and $K_{0.50}$ are the log-Pearson Type III deviates for exceedence probabilities of 0.01 and 0.50, respectively. The values of $Q_{0.01}$, $Q_{0.10}$ and $Q_{0.50}$ must usually be interpolated since probabilities computed with Equation 4.53 are not normally those needed to compute the properties of the synthetic or truncated distribution.

The log-Pearson Type III distribution can then be computed in the conventional manner using the synthetic statistical properties. USGS Bulletin 17B (1982) recommends the distribution be compared with the observed flows since data adjusted for conditional probability may not follow a log-Pearson Type III distribution.

4.3.6.4 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, with each record analyzed separately by an appropriate frequency distribution. The two separate frequency curves can then be combined through the concept of the addition of the probabilities of two events as follows:

$$Pr\,(Q \text{ or } Q_m) = Pr(Q) + Pr(Q_m) - Pr(Q)Pr(Q_m) \quad (4.58)$$

4.3.6.5 Two-Station Comparison

The objective of this method is to improve the mean and standard deviation of the logarithms at a short-record station (Y) using the statistics from a nearby long-record station (X). The method is from Appendix 7 of USGS Bulletin 17B (1982). The steps of the procedure depend on the nature of the records. Specifically, there are two cases: (1) the entire short record occurred during the duration of the long-record station, and (2) only part of the short record occurred during the duration of the long-record station. The following notation applies to the procedure:

N_x = record length at long-record station

N_1 = number of years when flows were concurrently observed at X and Y

N_2 = number of years when flows were observed at the long-record station, but not observed at the short-record station

N_3 = record length at short-record station

S_y = Standard deviation of the logarithm of flows for the extended period at the short-record station

Sx_1 = Standard deviation of logarithm of flows at the long-record station during the concurrent period

Sx_2 = Standard deviation of logarithm of flows at the long-record station for the period when flows were not observed at the short-record station

Sy_1 = Standard deviation of the logarithm of flows at the short-record station for the concurrent period

Sy_3 = Standard deviation of logarithm of flows for the entire period at the short-record station

X_1 = Logarithms of flows for the long-record station during the concurrent period

$\overline{X}_1$ = Mean logarithm of flows at the long-record station for the concurrent period

$\overline{X}_2$ = Mean logarithm of flows at the long-record station for the period when flow records are not available at the short-record station

$\overline{X}_3$ = Mean logarithm of flows for the entire period at the long-record station

Y_1 = Logarithms of flows for the short-record station during the concurrent period

$\overline{Y}$ = Mean logarithm of flows for the extended period at the short-record station

$\overline{Y}_1$ = Mean logarithm of flows for the period of observed flow at the short-record station (concurrent period)

$\overline{Y}_3$ = Mean logarithm of flows for the entire period at the short-record station

Case 1 is where N_1 equals N_3. Case 2 is where N_3 is greater than N_1.

The following procedure is used:

1a. Compute the regression coefficient, b:

$$b = \frac{\sum X_1 Y_1 - \sum X_1 \sum Y_1 / N_1}{\sum X_1^2 - \left(\sum X_1\right)^2 / N_1} \qquad (4.59)$$

1b. Compute the correlation coefficient, r:

$$r = b \frac{S_{x_1}}{S_{y_1}} \tag{4.60}$$

2. If Case 1 applies, go to Step 4; however, if case 2 applies, begin at Step 3.

3a. Compute the variance of the adjusted mean ($\bar{Y}$):

$$Var(\bar{Y}) = \frac{(S_{y_1})^2}{N_1}\left[1 - \frac{N_2}{N_1 + N_2}\left(r^2 - \frac{(1 - r^2)}{(N_1 - 3)}\right)\right] \tag{4.61}$$

3b. Compute S_{y3}^2:

$$S_{y3}^2 = \frac{1}{N_3 - 1}\sum_{i=1}^{N_3}(Y_i - \bar{Y}_3)^2 \tag{4.62}$$

3c. Compute the variance of the mean $\bar{Y}_3$ of the entire record at the short-record station:

$$var(\bar{Y}_3) = \frac{(S_{y_3})^2}{N_3} \tag{4.63}$$

3d. Compare Var($\bar{Y}$) and Var($\bar{Y}_3$). If Var($\bar{Y}$) < Var($\bar{Y}_3$), then go to Step 4; otherwise, go to Step 3e.

3e. Compute $\bar{Y}_3$, which should be used as the best estimate of the mean:

$$\bar{Y}_3 = \frac{1}{N_3}\sum_{i=1}^{N_3} Y_i \tag{4.64}$$

3f. Go to Step 5.

4a. Compute the critical correlation coefficient r_c:

$$r_c = \frac{1}{(N_1 - 2)^{0.5}} \tag{4.65}$$

4b. If $r > r_c$, then adjust the mean:

$$\bar{Y} = \bar{Y}_1 + \frac{N_2}{N_1 + N_2}\left[b\left(\bar{X}_2 - \bar{X}_1\right)\right] \tag{4.66a}$$

or

$$\bar{Y} = \bar{Y}_1 + b\left(\bar{X}_3 - \bar{X}_1\right) \tag{4.66b}$$

and go to Step 5.

4c. If $r \le r_c$, use $\bar{Y}_1$ for Case 1 or $\bar{Y}_3$ for Case 2 and go to Step 5.

5. If Case 1 applies, then go to Step 7; however, if Case 2 applies, begin at Step 6.

6a. Compute the variance of the adjusted variance S_y^2:

$$Var(S_Y^2) = \frac{2(S_{Y_1})^4}{N_1 - 1} + \frac{N_2(S_{Y_1})^4}{(N_1 + N_2 - 1)^2}\left[Ar^4 + Br^2 + C\right] \qquad (4.67)$$

where:

$$A = \frac{(N_2 + 2)(N_1 - 6)(N_1 - 8)}{(N_1 - 3)(N_1 - 5)} - \frac{8(N_1 - 4)}{(N_1 - 3)} - \frac{2N_2(N_1 - 4)^2}{(N_1 - 3)^2} + \frac{N_1 N_2 (N_1 - 4)^2}{(N_1 - 3)^2 (N_1 - 2)} + \frac{4(N_1 - 4)}{(N_1 - 3)}$$

$$B = \frac{6(N_2 + 2)(N_1 - 6)}{(N_1 - 3)(N_1 - 5)} + \frac{2(N_1^2 - N_1 - 14)}{(N_1 - 3)} + \frac{2N_2(N_1 - 4)(N_1 - 5)}{(N_1 - 3)^2} - \frac{2(N_1 - 4)(N_1 + 3)}{(N_1 - 3)} - \frac{2N_1 N_2 (N_1 - 4)^2}{(N_1 - 3)^2 (N_1 - 2)} \qquad (4.68)$$

$$C = \frac{2(N_1 + 1)}{N_1 - 3} + \frac{3(N_2 + 2)}{(N_1 - 3)(N_1 - 5)} - \frac{(N_1 + 1)(2N_1 + N_2 - 2)}{N_1 - 1} + \frac{2N_2(N_1 - 4)}{(N_1 - 3)^2} + \frac{2(N_1 - 4)(N_1 + 1)}{(N_1 - 3)} + \frac{N_1 N_2 (N_1 - 4)^2}{(N_1 - 3)^2 (N_1 - 2)}$$

6b. Compute the variance of the variance (S_{y3}^2) of the entire record at the short-record station:

$$Var\left(S_{Y3}^2\right) = \frac{2\left(S_{Y3}^2\right)}{N_3 - 1} \qquad (4.69)$$

6c. If $Var(S_{y3}^2) > Var(S_y^2)$, go to Step 7; otherwise, go to Step 6d.

6d. Use S_{y3} as the best estimate of the standard deviation.

6e. Go to Step 8.

7a. Compute the critical correlation coefficient r_a:

$$r_a = \left[\frac{-B \pm \sqrt{B^2 - 4AC}}{2A} \right]^{0.5} \tag{4.70}$$

where A, B, and C are defined in Step 6a.

7b. If $|r| > r_a$, then adjust the variance:

$$S_Y^2 = \frac{1}{N_1 + N_2 - 1} \times \left[(N_1 - 1) S_{Y1}^{\ 2} + (N_2 - 1) b^2 S_{X2}^{\ 2} + \frac{N_2 (N_1 - 4)(N_1 - 1)}{(N_1 - 3)(N_1 - 2)} (1 - r^2) S_{Y1}^{\ 2} + \frac{(N_1 N_2)}{N_1 + N_2} b^2 (\bar{X}_2 - \bar{X}_1)^2 \right] \tag{4.71}$$

and go to Step 8.

7c. If $|r| < r_a$, use S_{y1}^2 for Case 1 or S_{y3}^2 for Case 2 and go to Step 8.

8. The adjusted skew coefficient should be computed by weighting the generalized skew with the skew computed from the short-record station as described in USGS Bulletin 17B (1982).

<u>Example 4.11</u>. Table 4.22 contains flood series for two stations in SI and CU units, respectively. Forty-seven years of record are available at the long-record station (1912-1958). Thirty years of record are available at the short-record station (1929-1958). The logarithms of the data along with computed means and standard deviations are also provided in the table. The two-station comparison approach will be applied to improve the estimates of mean and standard deviation for the short-record station. Since the short-record station is a subset, in time, of the long-record station, the analysis is conducted using case 1.

Step 1 is to compute the correlation coefficient. The regression coefficient is calculated using Equation 4.59, as follows:

Variable	Value in SI	Value in CU
$b = \dfrac{\Sigma X_1 Y_1 - \Sigma X_1 \Sigma Y_1 / N_1}{\Sigma X_1^2 - (\Sigma X_1)^2 / N_1}$	$= \dfrac{177.04 - \dfrac{(82.23)(63.53)}{30}}{229.99 - (82.23)^2 / 30}$ $= 0.631$	$= \dfrac{474.55 - \dfrac{(128.67)(109.97)}{30}}{556.46 - (128.67)^2 / 30}$ $= 0.631$

Then, the correlation coefficient, r, is calculated using Equation 4.60. S_{X1} and S_{Y1} can be calculated from the data in Table 4.22 as 0.398 and 0.303, respectively.

Variable	Value in SI	Value in CU
$r = b\frac{S_{x_1}}{S_{y_1}}$	$= (0.631)\frac{0.398}{0.303} = 0.83$	$= (0.631)\frac{0.398}{0.303} = 0.83$

For case 1, the next step (step 4) is to compute the critical correlation coefficient, r_C, according to Equation 4.65 and compare it to the correlation coefficient, r.

Variable	Value in SI	Value in CU
$r_c = \frac{1}{(N_1 - 2)^{0.5}}$	$= \frac{1}{(30-2)^{0.5}} = 0.19$	$= \frac{1}{(30-2)^{0.5}} = 0.19$

Since r > r_C , the mean value of logarithms for the short-record station is adjusted using Equation 4.66a:

$$\overline{Y} = Y_1 + \frac{N_2}{N_1 + N_2}[b(\overline{X}_2 - \overline{X}_1)] = 2.118 + \frac{17}{30+17}[0.631(2.685 - 2.741)] = 2.105 \quad \text{(SI)}$$

$$\overline{Y} = Y_1 + \frac{N_2}{N_1 + N_2}[b(\overline{X}_2 - \overline{X}_1)] = 3.666 + \frac{17}{30+17}[0.631(4.233 - 4.289)] = 3.653 \quad \text{(CU)}$$

For case 1, the next step (step 7) is to compute the critical correlation coefficient, r_a, according to Equation 4.70 and compare it to the correlation coefficient, r. A, B, and C are –3.628, 0.4406, and 0.01472, respectively.

$$r_a = \left[\frac{-B \pm \sqrt{B^2 - 4AC}}{2A}\right]^{0.5} = \left(\frac{-0.4406 \pm \sqrt{(0.4406)^2 - 4(-3.628)(0.01472)}}{2(-3.628)}\right)^{0.5} = 0.39$$

Since $|r|$ > r_a, the variance of logarithms for the short-record station is adjusted using Equation 4.71, which gives an adjusted variance of 0.07957 and yields S_y= 0.282.

Improved estimates of the mean and standard deviation have been developed using the long-record data. A mean of 2.105 log (m^3/s) (3.653 log (ft^3/s)) supersedes a mean of 2.118 log (m^3/s) (3.666 log (ft^3/s)) while a standard deviation of 0.282 supersedes a standard deviation of 0.303. Step 8 is used to compute an adjusted skew. The revised mean and standard deviation along with the adjusted skew may now be applied to estimate design discharges.

4.3.7 Sequence of Flood Frequency Calculations

The above sections have discussed several standard frequency distributions and a variety of adjustments to test or 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 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. Some adjustments must be made before others can be made.

Unless there are compelling reasons not to use the log-Pearson Type III distribution, it should be used when making a flood frequency analysis. USGS Bulletin 17B (1982) presents a flow chart outlining a path through the frequency calculations and adjustments. This outline forms the basis for many of the available log-Pearson Type III computer programs.

Table 4.22(SI). Data for Two-Station Adjustment

	Long-record Station		**Short-record Station**			
Year	Flow (m^3/s)	Log Flow	Flow (m^3/s)	Log Flow	X_1Y_1	X_1^2
1912	129	2.111				
1913	220	2.342				
1914	918	2.963				
1915	779	2.892				
1916	538	2.731				
1917	680	2.833				
1918	374	2.573				
1919	439	2.642				
1920	289	2.461				
1921	399	2.601				
1922	419	2.622				
1923	297	2.473				
1924	326	2.513				
1925	779	2.892				
1926	504	2.702				
1927	1,028	3.012				
1928	1,914	3.282				
1929	156	2.193	43	1.633	3.582	4.810
1930	722	2.859	170	2.230	6.376	8.171
1931	158	2.199	42	1.623	3.569	4.834
1932	283	2.452	154	2.188	5.363	6.011
1933	144	2.158	31	1.491	3.219	4.659
1934	314	2.497	74	1.869	4.667	6.235
1935	722	2.859	114	2.057	5.880	8.171
1936	1,082	3.034	124	2.093	6.352	9.207
1937	224	2.350	94	1.973	4.637	5.524
1938	2,633	3.420	651	2.814	9.624	11.699
1939	91	1.959	36	1.556	3.049	3.838
1940	1,705	3.232	323	2.509	8.109	10.444
1941	858	2.933	346	2.539	7.448	8.605
1942	994	2.997	312	2.494	7.476	8.984
1943	1,537	3.187	197	2.294	7.312	10.155
1944	240	2.380	91	1.959	4.663	5.665
1945	810	2.908	91	1.959	5.698	8.459
1946	623	2.794	175	2.243	6.268	7.809
1947	504	2.702	115	2.061	5.569	7.303
1948	470	2.672	207	2.316	6.188	7.140
1949	174	2.241	110	2.041	4.574	5.020
1950	507	2.705	125	2.097	5.672	7.317

Table 4.22(SI). Data for Two-Station Adjustment

	Long-record Station		Short-record Station			
1951	1,421	3.153	110	2.041	6.436	9.939
1952	595	2.775	150	2.176	6.038	7.698
1953	1,133	3.054	218	2.338	7.142	9.328
1954	649	2.812	139	2.143	6.027	7.909
1955	167	2.223	70	1.845	4.101	4.940
1956	2,945	3.469	260	2.415	8.378	12.035
1957	926	2.967	174	2.241	6.647	8.801
1958	1,113	3.046	195	2.290	6.977	9.281
Total Record						
Sum		127.87		63.53	177.04	229.99
Mean		2.721		2.118		
Standard Deviation		0.357		0.303		
Concurrent Record						
Sum		82.23		63.53	177.04	229.99
Mean		2.741		2.118		
Standard Deviation		0.398		0.303		
Long Record Only						
Mean		2.685				

Table 4.22(CU). Data for Two-Station Adjustment

	Long-record Station		Short-record Station			
Year	Flow (ft^3/s)	Log Flow	Flow (ft^3/s)	Log Flow	X_1Y_1	X_1^2
1912	4,570	3.660				
1913	7,760	3.890				
1914	32,400	4.511				
1915	27,500	4.439				
1916	19,000	4.279				
1917	24,000	4.380				
1918	13,200	4.121				
1919	15,500	4.190				
1920	10,200	4.009				
1921	14,100	4.149				
1922	14,800	4.170				
1923	10,500	4.021				
1924	11,500	4.061				
1925	27,500	4.439				
1926	17,800	4.250				
1927	36,300	4.560				
1928	67,600	4.830				
1929	5,500	3.740	1,520	3.182	11.901	13.990
1930	25,500	4.407	6,000	3.778	16.649	19.418
1931	5,570	3.746	1,500	3.176	11.897	14.031

Table 4.22(CU). Data for Two-Station Adjustment

	Long-record Station		Short-record Station			
Year	Flow (ft^3/s)	Log Flow	Flow (ft^3/s)	Log Flow	X_1Y_1	X_1^2
1932	9,980	3.999	5,440	3.736	14.939	15.993
1933	5,100	3.708	1,080	3.033	11.247	13.746
1934	11,100	4.045	2,630	3.420	13.835	16.365
1935	25,500	4.407	4,010	3.603	15.877	19.418
1936	38,200	4.582	4,380	3.641	16.685	20.995
1937	7,920	3.899	3,310	3.520	13.723	15.200
1938	93,000	4.968	23,000	4.362	21.671	24.686
1939	3,230	3.509	1,260	3.100	10.880	12.315
1940	60,200	4.780	11,400	4.057	19.390	22.845
1941	30,300	4.481	12,200	4.086	18.313	20.083
1942	35,100	4.545	11,000	4.041	18.369	20.660
1943	54,300	4.735	6,970	3.843	18.197	22.418
1944	8,460	3.927	3,220	3.508	13.777	15.424
1945	28,600	4.456	3,230	3.509	15.638	19.859
1946	22,000	4.342	6,180	3.791	16.462	18.857
1947	17,800	4.250	4,070	3.610	15.342	18.066
1948	16,600	4.220	7,320	3.865	16.309	17.809
1949	6,140	3.788	3,870	3.588	13.591	14.350
1950	17,900	4.253	4,430	3.646	15.508	18.087
1951	50,200	4.701	3,870	3.588	16.865	22.097
1952	21,000	4.322	5,280	3.723	16.090	18.682
1953	40,000	4.602	7,710	3.887	17.888	21.179
1954	22,900	4.360	4,910	3.691	16.093	19.008
1955	5,900	3.771	2,480	3.394	12.800	14.219
1956	104,000	5.017	9,180	3.963	19.882	25.171
1957	32,700	4.515	6,140	3.788	17.102	20.381
1958	39,300	4.594	6,880	3.838	17.631	21.108
Total Record						
Sum		200.63		109.97	474.55	556.46
Mean		4.269		3.666		
Standard Deviation		0.357		0.303		
Concurrent Record						
Sum		128.67		109.97	474.55	556.46
Mean		4.289		3.666		
Standard Deviation		0.398		0.303		
Long Record Only						
Mean		4.233				

The SCS Handbook (1972) also outlines a sequence for flood frequency analysis that is summarized as follows:

1. Obtain site information, the systematic station data, and historic information. These 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 that best describes 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 are 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.

4.3.8 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 about the mean, and the skew as the third moment about the mean.

Three other recognized methods are used to determine frequency curves. They include the method of maximum likelihood, the L-moments or probability weighted moments, 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 (1988) and appropriate tables are presented from which the standard distributions can be determined.

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.

4.3.9 Low-flow Frequency Analysis

While instantaneous maximum discharges are used for flood frequency analyses, hydrologists are frequently interested in low flows. Low-flow frequency analyses are needed as part of water-quality studies and the design of culverts where fish passage is a design criterion. For low-flow frequency analyses, it is common to specify both a return period and a flow duration. For example, a low-flow frequency curve may be computed for a 7-day duration. In this case, the 10-year event would be referred to as the 7-day, 10-year low flow.

A data record to make a low-flow frequency analysis is compiled by identifying the lowest mean flow rate in each year of record for the given duration. For example, if the 21-day low-flow frequency curve is needed, the record for each year is analyzed to find the 21-day period in which the mean flow is the lowest. A moving-average smoothing analysis with a 21-day smoothing interval could be used to identify this flow. For a record of N years, such an analysis will yield N low flows for the needed duration.

The computational procedure for making a low-flow frequency analysis is very similar to that for a flood frequency analysis. It is first necessary to specify the probability distribution. The log-normal distribution is most commonly used, although another distribution could be used.

To make a log-normal analysis, a logarithmic transform of each of the N low flows is made. The mean and standard deviation of the logarithms are computed. Up to this point, the analysis is the same as for an analysis of peak flood flows. However, for a low-flow analysis, the governing equation is as follows:

$$log\, Y = \overline{Y}_L - z\, S_L \tag{4.72}$$

where,

$\overline{Y}_L$, S_L = logarithmic mean and standard deviation, respectively

z = standard normal deviate.

Note that Equation 4.73 includes a minus sign rather than the plus sign of Equation 4.27. Thus, the low-flow frequency curve will have a negative slope rather than the positive slope that is typical of peak-flow frequency curves. Also, computed low flows for the less frequent events (e.g., the 100-year low flow) will be less than the mean. For example, if the logarithmic statistics for a 7-day low-flow record are $\overline{Q}_L$ = 1.1 and S_L = 0.2, the 7-day, 50-year low flow is:

$$log\, Y = 1.1 - 2.054\,(0.2) = 0.6892$$

$$Q = 4.9\, m^3/s\ (\, 170\, ft^3/s\,)$$

To plot the data points so they can be compared with the computed population curve, the low flows are ranked from smallest to largest (not largest to smallest as with a peak-flow analysis). The smallest flow is given a rank of 1 and the largest flow is given a rank of N. A plotting position formula (Equation 4.21) can then be selected to compute the probabilities. Each magnitude is plotted against the corresponding probability. The probability is plotted on the upper horizontal axis and is interpreted as the probability that the flow in any one time period will be less than the value on the frequency curve. For the calculation provided above, there is a 2 percent chance that the 7-day mean flow will be less than 4.9 m^3/s (170 ft^3/s) in any one year.

4.4 INDEX ADJUSTMENT OF FLOOD RECORDS

The flood frequency methods of this chapter assume that the flood record is a series of events from the same population. In statistical terms, the events should be independent and identically distributed. In hydrologic terms, the events should be the result of the same meteorological and runoff processes. The year-to-year variation should only be due to the natural variation such as that of the volumes and durations of rainfall events.

Watershed changes, such as deforestation and urbanization, change the runoff processes that control the watershed response to rainfall. In statistical terms, the events are no longer identically distributed because the population changes with changes in land use. Afforestation might decrease the mean flow. Urbanization would probably increase the mean flow but decrease the variation of the peak discharges. If the watershed change takes place over an extended period, each event during the period of change is from a different population. Thus, magnitudes and exceedence probabilities obtained from the flood record could not represent future events. Before such a record is used for a frequency analysis, the measured events should be adjusted to reflect homogeneous watershed conditions. One method of adjusting a flood record is referred to as the index-adjustment method (which should not be confused with the index-flood method of Chapter 5).

Flood records can be adjusted using an index method, which is a class of methods that uses an index variable, such as the percentage of imperviousness or the fraction of a channel reach that has undergone channelization, to adjust the flood peaks. Index methods require values of the index variable for each year of the record and a model that relates the change in peak discharge, the index variable, and the exceedence probability. In addition to urbanization, index methods could be calibrated to adjust for the effects of deforestation, surface mining activity, agricultural management practices, or climate change.

4.4.1 Index Adjustment Method for Urbanization

Since urbanization is a common cause of nonhomogeneity in flood records, it will be used to illustrate index adjustment of floods. The literature does not identify a single method that is considered to be the best method for adjusting an annual flood series when only the time record of urbanization is available. Furthermore, urbanization may be defined by a number of parameters, which include, but are not limited to: percent imperviousness, percent urbanized land cover (residential, commercial, and industrial), and population density. Each method depends on the data used to calibrate the prediction process, and the data used to calibrate the methods are usually very sparse. However, the sensitivities of measured peak discharges suggest that a 1 percent increase in percent imperviousness causes an increase in peak discharge of about 1 to 2.5 percent for the 100-year and the 2-year events, respectively (McCuen, 1989).

Based on the general trends of results published in available urban flood-frequency studies, McCuen (1989) developed a method of adjusting a flood record for the effects of urbanization. Urbanization refers to the introduction of impervious surfaces or improvements of the hydraulic characteristics of the channels or principal flow paths. Figure 4.16 shows the peak adjustment factor as a function of the exceedence probability for percentages of imperviousness up to 60 percent. The greatest effect is for the more frequent events and the highest percentage of imperviousness. For this discussion, percent imperviousness is used as the measure of urbanization.

Given the return period of a flood peak for a nonurbanized watershed, the effect of an increase in urbanization can be assessed by multiplying the discharge by the peak adjustment factor, which is a function of the return period and the percentage of urbanization. Where it is necessary to adjust a discharge to another watershed condition, the measured discharge can be divided by the peak adjustment factor for the existing condition to produce a "rural" discharge. This computed discharge is then multiplied by the peak adjustment factor for the second watershed condition. The first operation (i.e., division) adjusts the discharge to a magnitude representative of a nonurbanized condition while the second operation (i.e., multiplication) adjusts the new discharge to a computed discharge for the second watershed condition.

4.4.2 Adjustment Procedure

The adjustment method of Figure 4.16 requires an exceedence probability. For a flood record, the best estimate of the probability is obtained from a plotting position formula.

The following procedures can be used to adjust a flood record for which the individual flood events have occurred on a watershed that is undergoing a continuous change in the level of urbanization:

1. Identify the percentage of urbanization for each event in the flood record. While percentages may not be available for every year of record, they will have to be interpolated or extrapolated from existing estimates so a percentage is assigned to each flood event of record.

2. Identify the percentage of urbanization for which an adjusted flood record is needed. This is the percentage to which all flood events in the record will be adjusted, thus producing a record that is assumed to include events that are independent and identically distributed.

3. Compute the rank (i) and exceedence probability (p) for each event in the flood record; a plotting position formula can be used to compute the probability.

4. Using the exceedence probability and the percentage of urbanization from Step 1, find the peak adjustment factor (f_1) from Figure 4.16 to transform the measured peak from the actual level of urbanization to a nonurbanized condition.

5. Using the exceedence probability and the percentage of urbanization from Step 2 for which a flood series is needed from Figure 4.16, find the peak adjustment factor (f_2) that is necessary to transform the computed nonurbanized peak of Step 4 to a discharge for the desired level of urbanization.

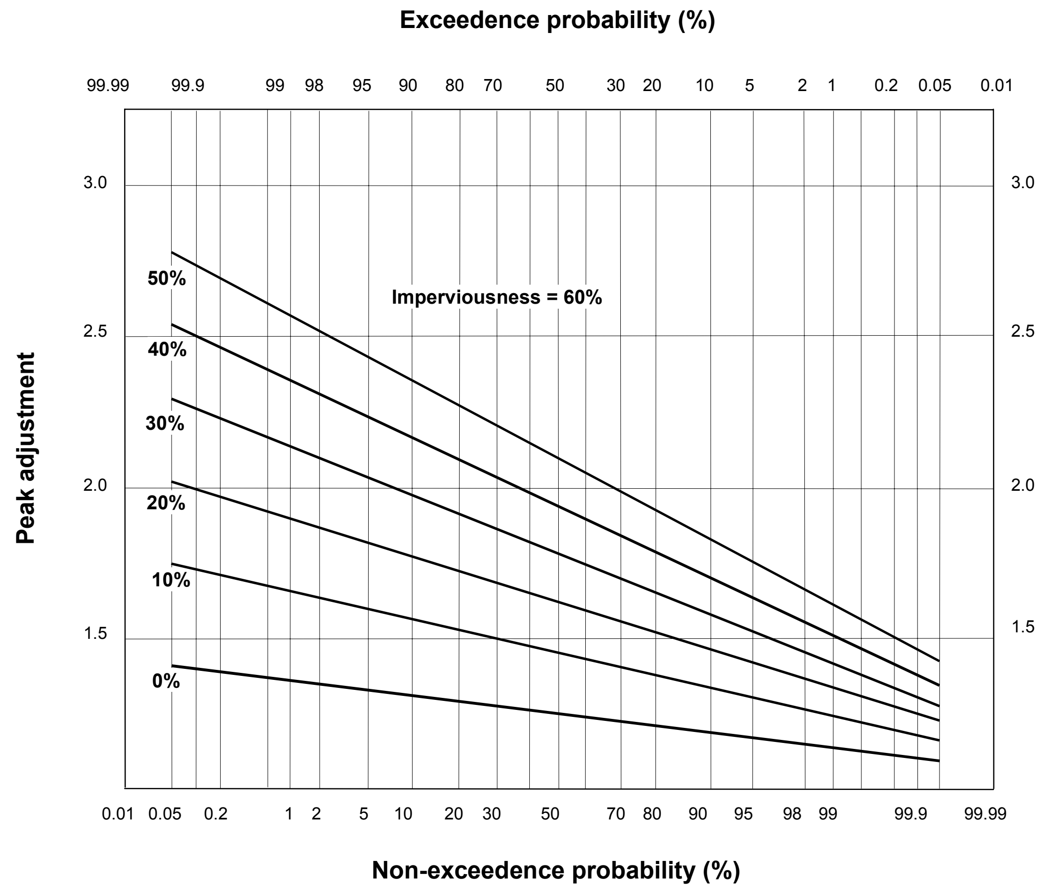


Figure 4.16. Peak adjustment factors for correcting a flood discharge magnitude for the change in imperviousness (from McCuen, 1989)

6. Compute the adjusted discharge (Q_a) by:

$$Q_a = \frac{f_2}{f_1} Q \qquad (4.73)$$

in which Q is the measured discharge.

7. Repeat Steps 4, 5, and 6 for each event in the flood record and rank the adjusted series.

8. If the ranks of the events in the adjusted series differ from the ranks of the previous series, which would be the measured events after one iteration of Steps 3 to 7, then the iteration process should be repeated until the ranks do not change.

Example 4.12. Table 4.23 (SI) and Table 4.23 (CU) contain the 48-year record of annual maximum peak discharges for the Rubio Wash watershed in Los Angeles in SI and CU units, respectively. Between 1929 and 1964, the percent of impervious cover, which is also given in Table 4.23, increased from 18 to 40 percent. The log moments are summarized below.

Variable	Value in SI	Value in CU
Log mean	1.704	3.252
Log standard deviation	0.191	0.191
Station skew	-0.7	-0.7
Generalized skew	-0.45	-0.45

The procedure given above was used to adjust the flood record for the period from 1929 to 1963 to current impervious cover conditions. For example, while the peak discharges for 1931 and 1945 occurred when the percent impervious cover was 19 and 34 percent, respectively, the values were adjusted to a common percentage of 40 percent, which is the watershed state after 1964. For this example, imperviousness was used as the index variable as a measure of urbanization.

The adjusted rank after each iteration is compared with the rank prior to the iteration to determine if the computations are complete. If changes occur, a subsequent iteration may be required. Three iterations of adjustments were required for this example. The iterative process is required because the ranks for some of the earlier events changed considerably from the ranks of the measured record; for example, the rank of the 1930 peak changed from 30 to 22 on the first trial, and the rank of the 1933 event went from 20 to 14. Because of such changes in the rank, the exceedence probabilities change and thus the adjustment factors, which depend on the exceedence probabilities, change. After the third adjustment is made, the rank of the events did not change, so the process is complete. The adjusted series is given in Table 4.23.
The adjusted series has a mean and standard deviation of 1.732 and 0.179, respectively, in SI units (3.280 and 0.178 in CU units). The mean increased, but the standard deviation decreased. Thus the adjusted flood frequency curve will, in general, be higher than the curve for the measured series, but will have a small slope. The computations for the adjusted and unadjusted flood frequency curves are given in Table 4.24 (SI) and Table 4.24 (CU). Since the measured series was not homogeneous, the generalized skew of -0.45 was used to compute the values for the flood frequency curve. The percent increase in the 2-, 5-, 10-, 25-, 50- and 100-year flood magnitudes are also given in Table 4.24. The change is relatively minor because the imperviousness did not change after 1964 and the change was small (i.e., 10 percent) from 1942 to 1964; also most of the larger storm events occurred after the watershed had reached the developed condition. The adjusted series would represent the annual flood series for a constant urbanization condition (i.e., 40 percent imperviousness). Of course, the adjusted series is not a measured series.

Table 4.23(SI). Adjustment of the Rubio Wash Annual Maximum Flood Record for Urbanization

				Iteration 1				
Year	Impervious-ness (%)	Measured Discharge (m^3/s)	Rank	Exceedence Probability	f_1	f_2	Adjusted Discharge (m^3/s)	Adjusted Rank
1929	18	18.7	47	0.959	1.560	2.075	24.9	47
1930	18	47.8	30	0.612	1.434	1.846	61.5	22
1931	19	22.6	46	0.939	1.573	2.044	29.4	44
1932	20	42.8	34	0.694	1.503	1.881	53.6	32
1933	20	58.6	20	0.408	1.433	1.765	72.2	13
1934	21	47.6	31	0.633	1.506	1.855	58.6	24
1935	21	38.8	35	0.714	1.528	1.890	48.0	34
1936	22	33.4	40	0.816	1.589	1.956	41.1	36
1937	23	68.0	14	0.286	1.448	1.713	80.4	8
1938	25	48.7	29	0.592	1.568	1.838	57.1	28
1939	26	28.3	43	0.878	1.690	1.984	33.2	42
1940	28	54.9	26	0.531	1.603	1.814	62.1	20
1941	29	34.0	38	0.776	1.712	1.931	38.3	37
1942	30	78.7	7	0.143	1.508	1.648	86.0	5
1943	31	54.6	27	0.551	1.663	1.822	59.8	23
1944	33	50.4	28	0.571	1.705	1.830	54.1	31
1945	34	46.1	32	0.653	1.752	1.863	49.0	33
1946	34	75.0	10	0.204	1.585	1.672	79.1	10
1947	35	59.2	19	0.388	1.675	1.757	62.1	21
1948	36	15.0	48	0.980	2.027	2.123	15.7	48
1949	37	30.0	42	0.857	1.907	1.969	31.0	43
1950	38	64.8	17	0.347	1.708	1.740	66.0	16
1951	38	85.5	4	0.082	1.557	1.583	86.9	4
1952	39	62.3	18	0.367	1.732	1.748	62.9	19
1953	39	65.4	15	0.306	1.706	1.722	66.0	17
1954	39	36.5	36	0.735	1.881	1.900	36.9	38
1955	39	55.8	25	0.510	1.788	1.806	56.4	29
1956	39	84.4	5	0.102	1.589	1.602	85.1	6
1957	39	77.6	9	0.184	1.646	1.660	78.3	11
1958	39	78.7	8	0.163	1.620	1.634	79.4	9
1959	39	27.9	44	0.898	1.979	2.001	28.2	45
1960	39	25.5	45	0.918	1.999	2.020	25.8	46
1961	39	34.0	39	0.796	1.911	1.931	34.4	40
1962	39	33.4	41	0.837	1.935	1.956	33.8	41
1963	39	44.5	33	0.673	1.853	1.872	45.0	35
1964	40	57.8	22	0.449	1.781	1.781	57.8	27
1965	40	65.1	16	0.327	1.731	1.731	65.1	18
1966	40	57.8	23	0.469	1.790	1.790	57.8	26
1967	40	69.6	13	0.265	1.703	1.703	69.6	15
1968	40	81.8	6	0.122	1.619	1.619	81.8	7
1969	40	71.9	12	0.245	1.693	1.693	71.9	14
1970	40	104.8	1	0.020	1.480	1.480	104.8	1
1971	40	35.1	37	0.755	1.910	1.910	35.1	39
1972	40	89.6	3	0.061	1.559	1.559	89.6	3
1973	40	56.2	24	0.490	1.798	1.798	56.2	30
1974	40	90.0	2	0.041	1.528	1.528	90.0	2
1975	40	58.6	21	0.429	1.773	1.773	58.6	25
1976	40	73.9	11	0.224	1.683	1.683	73.9	12

Table 4.23(SI). Adjustment of the Rubio Wash Annual Maximum Flood Record for Urbanization (cont'd)

Year	Impervious-ness (%)	Measured Discharge (m^3/s)	Adjusted Rank-Iteration 1	Iteration 2: Adjusted Exceedence Probability	Iteration 2: f_1	Iteration 2: f_2	Iteration 2: Adjusted Discharge (m^3/s)	Adjusted Rank-Iteration 2
1929	18	18.7	47	0.959	1.560	2.075	24.9	47
1930	18	47.8	22	0.449	1.399	1.781	60.9	22
1931	19	22.6	44	0.898	1.548	2.001	29.2	44
1932	20	42.8	32	0.653	1.493	1.863	53.4	32
1933	20	58.6	13	0.265	1.395	1.703	71.5	14
1934	21	47.6	24	0.490	1.475	1.806	58.3	25
1935	21	38.8	34	0.694	1.522	1.881	48.0	34
1936	22	33.4	36	0.735	1.553	1.900	40.9	36
1937	23	68.0	8	0.163	1.405	1.648	79.8	8
1938	25	48.7	28	0.571	1.562	1.830	57.1	28
1939	26	28.3	42	0.857	1.680	1.969	33.2	42
1940	28	54.9	20	0.408	1.573	1.773	61.9	21
1941	29	34.0	37	0.755	1.695	1.910	38.3	37
1942	30	78.7	5	0.102	1.472	1.602	85.7	5
1943	31	54.6	23	0.469	1.637	1.790	59.7	23
1944	33	50.4	31	0.633	1.726	1.855	54.2	31
1945	34	46.1	33	0.673	1.760	1.872	49.0	33
1946	34	75.0	10	0.204	1.585	1.672	79.1	10
1947	35	59.2	21	0.429	1.690	1.773	62.1	20
1948	36	15.0	48	0.980	2.027	2.123	15.7	48
1949	37	30.0	43	0.878	1.921	1.984	31.0	43
1950	38	64.8	16	0.327	1.708	1.740	66.0	16
1951	38	85.5	4	0.082	1.557	1.583	86.9	4
1952	39	62.3	19	0.388	1.741	1.757	62.9	19
1953	39	65.4	17	0.347	1.724	1.740	66.0	17
1954	39	36.5	38	0.776	1.901	1.920	36.9	38
1955	39	55.8	29	0.592	1.820	1.838	56.4	29
1956	39	84.4	6	0.122	1.606	1.619	85.1	6
1957	39	77.6	11	0.224	1.668	1.683	78.3	11
1958	39	78.7	9	0.184	1.646	1.660	79.4	9
1959	39	27.9	45	0.918	1.999	2.020	28.2	45
1960	39	25.5	46	0.939	2.022	2.044	25.8	46
1961	39	34.0	40	0.816	1.923	1.943	34.4	40
1962	39	33.4	41	0.837	1.935	1.956	33.8	41
1963	39	44.5	35	0.714	1.871	1.890	45.0	35
1964	40	57.8	27	0.551	1.822	1.822	57.8	26
1965	40	65.1	18	0.367	1.748	1.748	65.1	18
1966	40	57.8	26	0.531	1.822	1.822	57.8	27
1967	40	69.6	15	0.306	1.722	1.722	69.6	15
1968	40	81.8	7	0.143	1.634	1.634	81.8	7
1969	40	71.9	14	0.286	1.713	1.713	71.9	13
1970	40	104.8	1	0.020	1.480	1.480	104.8	1
1971	40	35.1	39	0.796	1.931	1.931	35.1	39
1972	40	89.6	3	0.061	1.559	1.559	89.6	3
1973	40	56.2	30	0.612	1.846	1.846	56.2	30
1974	40	90.0	2	0.041	1.528	1.528	90.0	2
1975	40	58.6	25	0.510	1.806	1.806	58.6	24
1976	40	73.9	12	0.245	1.693	1.693	73.9	12

Table 4.23(SI). Adjustment of the Rubio Wash Annual Maximum Flood Record for Urbanization (cont'd)

Year	Impervious-ness (%)	Measured Discharge (m^3/s)	Adjusted Rank-Iteration 2	Iteration 3: Adjusted Exceedence Probability	Iteration 3: f_1	Iteration 3: f_2	Iteration 3: Adjusted Discharge (m^3/s)	Adjusted Rank-Iteration 3
1929	18	18.7	47	0.959	1.560	2.075	24.9	47
1930	18	47.8	22	0.449	1.399	1.781	60.9	22
1931	19	22.6	44	0.898	1.548	2.001	29.2	44
1932	20	42.8	32	0.653	1.493	1.863	53.4	32
1933	20	58.6	14	0.286	1.401	1.713	71.7	14
1934	21	47.6	25	0.510	1.475	1.806	58.3	25
1935	21	38.8	34	0.694	1.522	1.881	48.0	34
1936	22	33.4	36	0.735	1.553	1.900	40.9	36
1937	23	68.0	8	0.163	1.405	1.648	79.8	8
1938	25	48.7	28	0.571	1.562	1.830	57.1	28
1939	26	28.3	42	0.857	1.680	1.969	33.2	42
1940	28	54.9	21	0.429	1.573	1.773	61.9	21
1941	29	34.0	37	0.755	1.695	1.910	38.3	37
1942	30	78.7	5	0.102	1.472	1.602	85.7	5
1943	31	54.6	23	0.469	1.637	1.790	59.7	23
1944	33	50.4	31	0.633	1.726	1.855	54.2	31
1945	34	46.1	33	0.673	1.760	1.872	49.0	33
1946	34	75.0	10	0.204	1.585	1.672	79.1	10
1947	35	59.2	20	0.408	1.683	1.765	62.1	20
1948	36	15.0	48	0.980	2.027	2.123	15.7	48
1949	37	30.0	43	0.878	1.921	1.984	31.0	43
1950	38	64.8	16	0.327	1.708	1.740	66.0	16
1951	38	85.5	4	0.082	1.557	1.583	86.9	4
1952	39	62.3	19	0.388	1.741	1.757	62.9	19
1953	39	65.4	17	0.347	1.724	1.740	66.0	17
1954	39	36.5	38	0.776	1.901	1.920	36.9	38
1955	39	55.8	29	0.592	1.820	1.838	56.4	29
1956	39	84.4	6	0.122	1.606	1.619	85.1	6
1957	39	77.6	11	0.224	1.668	1.683	78.3	11
1958	39	78.7	9	0.184	1.646	1.660	79.4	9
1959	39	27.9	45	0.918	1.999	2.020	28.2	45
1960	39	25.5	46	0.939	2.022	2.044	25.8	46
1961	39	34.0	40	0.816	1.923	1.943	34.4	40
1962	39	33.4	41	0.837	1.935	1.956	33.8	41
1963	39	44.5	35	0.714	1.871	1.890	45.0	35
1964	40	57.8	26	0.531	1.822	1.822	57.8	26
1965	40	65.1	18	0.367	1.748	1.748	65.1	18
1966	40	57.8	27	0.551	1.822	1.822	57.8	27
1967	40	69.6	15	0.306	1.722	1.722	69.6	15
1968	40	81.8	7	0.143	1.634	1.634	81.8	7
1969	40	71.9	13	0.265	1.703	1.703	71.9	13
1970	40	104.8	1	0.020	1.480	1.480	104.8	1
1971	40	35.1	39	0.796	1.931	1.931	35.1	39
1972	40	89.6	3	0.061	1.559	1.559	89.6	3
1973	40	56.2	30	0.612	1.846	1.846	56.2	30
1974	40	90.0	2	0.041	1.528	1.528	90.0	2
1975	40	58.6	24	0.490	1.798	1.798	58.6	24
1976	40	73.9	12	0.245	1.693	1.693	73.9	12

Table 4.23(CU). Adjustment of the Rubio Wash Annual Maximum Flood Record for Urbanization

				Iteration 1				
Year	Imperviousness (%)	Measured Discharge (ft^3/s)	Rank	Exceedence Probability	f_1	f_2	Adjusted Discharge (ft^3/s)	Adjusted Rank
1929	18	660	47	0.959	1.560	2.075	878	47
1930	18	1,690	30	0.612	1.434	1.846	2,176	22
1931	19	800	46	0.939	1.573	2.044	1,040	44
1932	20	1,510	34	0.694	1.503	1.881	1,890	32
1933	20	2,070	20	0.408	1.433	1.765	2,550	13
1934	21	1,680	31	0.633	1.506	1.855	2,069	24
1935	21	1,370	35	0.714	1.528	1.890	1,695	34
1936	22	1,180	40	0.816	1.589	1.956	1,453	36
1937	23	2,400	14	0.286	1.448	1.713	2,839	8
1938	25	1,720	29	0.592	1.568	1.838	2,016	28
1939	26	1,000	43	0.878	1.690	1.984	1,174	42
1940	28	1,940	26	0.531	1.603	1.814	2,195	20
1941	29	1,200	38	0.776	1.712	1.931	1,354	37
1942	30	2,780	7	0.143	1.508	1.648	3,038	5
1943	31	1,930	27	0.551	1.663	1.822	2,115	23
1944	33	1,780	28	0.571	1.705	1.830	1,910	31
1945	34	1,630	32	0.653	1.752	1.863	1,733	33
1946	34	2,650	10	0.204	1.585	1.672	2,795	10
1947	35	2,090	19	0.388	1.675	1.757	2,192	21
1948	36	530	48	0.980	2.027	2.123	555	48
1949	37	1,060	42	0.857	1.907	1.969	1,094	43
1950	38	2,290	17	0.347	1.708	1.740	2,333	16
1951	38	3,020	4	0.082	1.557	1.583	3,070	4
1952	39	2,200	18	0.367	1.732	1.748	2,220	19
1953	39	2,310	15	0.306	1.706	1.722	2,332	17
1954	39	1,290	36	0.735	1.881	1.900	1,303	38
1955	39	1,970	25	0.510	1.788	1.806	1,990	29
1956	39	2,980	5	0.102	1.589	1.602	3,004	6
1957	39	2,740	9	0.184	1.646	1.660	2,763	11
1958	39	2,780	8	0.163	1.620	1.634	2,804	9
1959	39	990	44	0.898	1.979	2.001	1,001	45
1960	39	900	45	0.918	1.999	2.020	909	46
1961	39	1,200	39	0.796	1.911	1.931	1,213	40
1962	39	1,180	41	0.837	1.935	1.956	1,193	41
1963	39	1,570	33	0.673	1.853	1.872	1,586	35
1964	40	2,040	22	0.449	1.781	1.781	2,040	27
1965	40	2,300	16	0.327	1.731	1.731	2,300	18
1966	40	2,040	23	0.469	1.790	1.790	2,040	26
1967	40	2,460	13	0.265	1.703	1.703	2,460	15
1968	40	2,890	6	0.122	1.619	1.619	2,890	7
1969	40	2,540	12	0.245	1.693	1.693	2,540	14
1970	40	3,700	1	0.020	1.480	1.480	3,700	1
1971	40	1,240	37	0.755	1.910	1.910	1,240	39
1972	40	3,160	3	0.061	1.559	1.559	3,160	3
1973	40	1,980	24	0.490	1.798	1.798	1,980	30
1974	40	3,180	2	0.041	1.528	1.528	3,180	2
1975	40	2,070	21	0.429	1.773	1.773	2,070	25
1976	40	2,610	11	0.224	1.683	1.683	2,610	12

Table 4.23(CU). Adjustment of the Rubio Wash Annual Maximum Flood Record for Urbanization (cont'd)

Year	Impervious-ness (%)	Measured Discharge (ft^3/s)	Adjusted Rank	Iteration 2: Adjusted Exceedence Probability	Iteration 2: f_1	Iteration 2: f_2	Iteration 2: Adjusted Discharge (ft^3/s)	Adjusted Rank
1929	18	660	47	0.959	1.560	2.075	878	47
1930	18	1,690	22	0.449	1.399	1.781	2,151	22
1931	19	800	44	0.898	1.548	2.001	1,034	44
1932	20	1,510	32	0.653	1.493	1.863	1,884	32
1933	20	2,070	13	0.265	1.395	1.703	2,527	14
1934	21	1,680	24	0.490	1.475	1.806	2,057	25
1935	21	1,370	34	0.694	1.522	1.881	1,693	34
1936	22	1,180	36	0.735	1.553	1.900	1,444	36
1937	23	2,400	8	0.163	1.405	1.648	2,815	8
1938	25	1,720	28	0.571	1.562	1.830	2,015	28
1939	26	1,000	42	0.857	1.680	1.969	1,172	42
1940	28	1,940	20	0.408	1.573	1.773	2,187	21
1941	29	1,200	37	0.755	1.695	1.910	1,352	37
1942	30	2,780	5	0.102	1.472	1.602	3,026	5
1943	31	1,930	23	0.469	1.637	1.790	2,110	23
1944	33	1,780	31	0.633	1.726	1.855	1,913	31
1945	34	1,630	33	0.673	1.760	1.872	1,734	33
1946	34	2,650	10	0.204	1.585	1.672	2,795	10
1947	35	2,090	21	0.429	1.690	1.773	2,193	20
1948	36	530	48	0.980	2.027	2.123	555	48
1949	37	1,060	43	0.878	1.921	1.984	1,095	43
1950	38	2,290	16	0.327	1.708	1.740	2,333	16
1951	38	3,020	4	0.082	1.557	1.583	3,070	4
1952	39	2,200	19	0.388	1.741	1.757	2,220	19
1953	39	2,310	17	0.347	1.724	1.740	2,331	17
1954	39	1,290	38	0.776	1.901	1.920	1,303	38
1955	39	1,970	29	0.592	1.820	1.838	1,989	29
1956	39	2,980	6	0.122	1.606	1.619	3,004	6
1957	39	2,740	11	0.224	1.668	1.683	2,765	11
1958	39	2,780	9	0.184	1.646	1.660	2,804	9
1959	39	990	45	0.918	1.999	2.020	1,000	45
1960	39	900	46	0.939	2.022	2.044	910	46
1961	39	1,200	40	0.816	1.923	1.943	1,212	40
1962	39	1,180	41	0.837	1.935	1.956	1,193	41
1963	39	1,570	35	0.714	1.871	1.890	1,586	35
1964	40	2,040	27	0.551	1.822	1.822	2,040	26
1965	40	2,300	18	0.367	1.748	1.748	2,300	18
1966	40	2,040	26	0.531	1.822	1.822	2,040	27
1967	40	2,460	15	0.306	1.722	1.722	2,460	15
1968	40	2,890	7	0.143	1.634	1.634	2,890	7
1969	40	2,540	14	0.286	1.713	1.713	2,540	13
1970	40	3,700	1	0.020	1.480	1.480	3,700	1
1971	40	1,240	39	0.796	1.931	1.931	1,240	39
1972	40	3,160	3	0.061	1.559	1.559	3,160	3
1973	40	1,980	30	0.612	1.846	1.846	1,980	30
1974	40	3,180	2	0.041	1.528	1.528	3,180	2
1975	40	2,070	25	0.510	1.806	1.806	2,070	24
1976	40	2,610	12	0.245	1.693	1.693	2,610	12

Table 4.23(CU) Adjustment of the Rubio Wash Annual Maximum Flood Record for Urbanization (cont'd)

Year	Impervious-ness (%)	Measured Discharge (ft³/s)	Adjusted Rank-iteration 2	Iteration 3 Adjusted Exceedence Probability	Iteration 3 f_1	Iteration 3 f_2	Iteration 3 Adjusted Discharge (ft³/s)	Adjusted Rank-iteration 3
1929	18	660	47	0.959	1.560	2.075	878	47
1930	18	1,690	22	0.449	1.399	1.781	2,151	22
1931	19	800	44	0.898	1.548	2.001	1,034	44
1932	20	1,510	32	0.653	1.493	1.863	1,884	32
1933	20	2,070	14	0.286	1.401	1.713	2,531	14
1934	21	1,680	25	0.510	1.475	1.806	2,057	25
1935	21	1,370	34	0.694	1.522	1.881	1,693	34
1936	22	1,180	36	0.735	1.553	1.900	1,444	36
1937	23	2,400	8	0.163	1.405	1.648	2,815	8
1938	25	1,720	28	0.571	1.562	1.830	2,015	28
1939	26	1,000	42	0.857	1.680	1.969	1,172	42
1940	28	1,940	21	0.429	1.573	1.773	2,187	21
1941	29	1,200	37	0.755	1.695	1.910	1,352	37
1942	30	2,780	5	0.102	1.472	1.602	3,026	5
1943	31	1,930	23	0.469	1.637	1.790	2,110	23
1944	33	1,780	31	0.633	1.726	1.855	1,913	31
1945	34	1,630	33	0.673	1.760	1.872	1,734	33
1946	34	2,650	10	0.204	1.585	1.672	2,795	10
1947	35	2,090	20	0.408	1.683	1.765	2,192	20
1948	36	530	48	0.980	2.027	2.123	555	48
1949	37	1,060	43	0.878	1.921	1.984	1,095	43
1950	38	2,290	16	0.327	1.708	1.740	2,333	16
1951	38	3,020	4	0.082	1.557	1.583	3,070	4
1952	39	2,200	19	0.388	1.741	1.757	2,220	19
1953	39	2,310	17	0.347	1.724	1.740	2,331	17
1954	39	1,290	38	0.776	1.901	1.920	1,303	38
1955	39	1,970	29	0.592	1.820	1.838	1,989	29
1956	39	2,980	6	0.122	1.606	1.619	3,004	6
1957	39	2,740	11	0.224	1.668	1.683	2,765	11
1958	39	2,780	9	0.184	1.646	1.660	2,804	9
1959	39	990	45	0.918	1.999	2.020	1,000	45
1960	39	900	46	0.939	2.022	2.044	910	46
1961	39	1,200	40	0.816	1.923	1.943	1,212	40
1962	39	1,180	41	0.837	1.935	1.956	1,193	41
1963	39	1,570	35	0.714	1.871	1.890	1,586	35
1964	40	2,040	26	0.531	1.822	1.822	2,040	26
1965	40	2,300	18	0.367	1.748	1.748	2,300	18
1966	40	2,040	27	0.551	1.822	1.822	2,040	27
1967	40	2,460	15	0.306	1.722	1.722	2,460	15
1968	40	2,890	7	0.143	1.634	1.634	2,890	7
1969	40	2,540	13	0.265	1.703	1.703	2,540	13
1970	40	3,700	1	0.020	1.480	1.480	3,700	1
1971	40	1,240	39	0.796	1.931	1.931	1,240	39
1972	40	3,160	3	0.061	1.559	1.559	3,160	3
1973	40	1,980	30	0.612	1.846	1.846	1,980	30
1974	40	3,180	2	0.041	1.528	1.528	3,180	2
1975	40	2,070	24	0.490	1.798	1.798	2,070	24
1976	40	2,610	12	0.245	1.693	1.693	2,610	12

Table 4.24(SI). Computed Discharges for Log-Pearson Type III (LP3) with Generalized Skew for Measured Series and Series Adjusted to 40 Percent Imperviousness

(1) Return period (yrs)	(2) LP3 deviate, K, for g = -0.45	(3) Discharges based on: Measured series(m^3/s)	(4) Adjusted series (m^3/s)	(5) Increase (%)
2	0.07476	52	56	8
5	0.85580	74	77	4
10	1.22366	87	89	2
25	1.58657	102	104	2
50	1.80538	112	114	2
100	1.99202	121	123	2

(3) $Q = 10^{1.704 + 0.191 K}$

(4) $Q = 10^{1.732 + 0.179K}$

Table 4.24(CU). Computed Discharges for Log-Pearson Type III (LP3) with Generalized Skew for Measured Series and Series Adjusted to 40 Percent Imperviousness

(1) Return period (yrs)	(2) LP3 deviate, K, for G = -0.45	(3) Discharges based on: Measured series (ft^3/s)	(4) Adjusted series (ft^3/s)	(5) Increase (%)
2	0.07476	1,850	1,960	6
5	0.85580	2,600	2,710	4
10	1.22366	3,060	3,150	3
25	1.58657	3,590	3,650	2
50	1.80538	3,950	3,990	1
100	1.99202	4,290	4,310	0

(3) $Q = 10^{3.252 + 0.191 K}$

(4) $Q = 10^{3.280 + 0.179K}$

4.5 PEAK FLOW TRANSPOSITION

Gaged flow data may be applied at design locations near, but not coincident with, the gage location using peak flow transposition. Peak flow transposition is the process of adjusting the peak flow determined at the gage to a downstream or upstream location. Peak flow transposition may also be accomplished if the design location is between two gages through an interpolation process.

The design location should be located on the same stream channel near the gage with no major tributaries draining to the channel in the intervening reach. The definition of "near" depends on the method applied and the changes in the contributing watershed between the gage and the design location.

Two methods of peak flow transposition have been commonly applied: the area-ratio method and the Sauer method (Sauer, 1973). The area-ratio method is described as:

$$Q_d = Q_g \left(\frac{A_d}{A_g} \right)^c \tag{4.74}$$

where,

Q_d = peak flow at the design location
Q_g = peak flow at the gage location
A_d = watershed area at the design location
A_g = watershed area at the gage location
c = transposition exponent.

Equation 4.74 is limited to design locations with drainage areas within 25 percent of the gage drainage area. The transposition exponent is frequently taken as the exponent for watershed area in an applicable peak flow regression equation for the site and is generally less than 1. (See Chapter 5 for more information on peak flow regression equations.)

In an evaluation by McCuen and Levy (2000), Sauer's method performed slightly better than the area-ratio method when tested on data from seven states for the 10- and 100-year events. Sauer's method is based first on computing a weighted discharge at the gage from the log-Pearson Type III analysis of the gage record and the regression equation estimate at the gage location. Then, Sauer uses the gage drainage area, the design location drainage area, the weighted gage discharge, and regression equation estimates at the gage and design locations to determine the appropriate flow at the design location. More detailed descriptions of Sauer's method are found in Sauer (1973) and McCuen and Levy (2000).

4.6 RISK ASSESSMENT

A measured flood record is the result of rainfall events that are considered randomly distributed. As such, the same rainfall record will not repeat itself and so future floods will be different from past floods. However, if the watershed remains unchanged, future floods are expected to be from the same population as past floods and, thus, have the same characteristics. The variation of future floods from past floods is referred to as sampling uncertainty.

Even if the true or correct probability distribution and the correct parameter values to use in computing a flood frequency curve were known, there is no certainty about the occurrence of floods over the design life of an engineering structure. A culvert might be designed to pass the 10-year flood (i.e., the flood having an exceedence probability of 0.1), but over any period of 10 years, the capacity may be reached as many as 10 times or not at all. A coffer dam constructed to withstand up to the 50-year flood may be exceeded shortly after being constructed, even though the dam will only be in place for 1 year. These are chance occurrences that are independent of the lack of knowledge of the true probability distribution. That is, the risk would occur even if we knew the true population of floods. Such risk of failure, or design uncertainty, can be estimated using the concept of binomial risk.

4.6.1 Binomial Distribution

The binomial distribution is used to define probabilities of discrete events; it is applicable to random variables that satisfy the following four assumptions:

1. There are *n* occurrences, or trials, of the random variable.

2. The *n* trials are independent.

3. There are only two possible outcomes for each trial.

4. The probability of each outcome is constant from trial to trial.

The probabilities of occurrence of any random variable satisfying these four assumptions can be computed using the binomial distribution. For example, if the random variable is defined as the annual occurrence or nonoccurrence of a flood of a specified magnitude, the binomial distribution is applicable. There are only two possible outcomes: the flood either occurs or does not occur. For the design life of a project of n years, there will be n possible occurrences and the n occurrences are independent of each other (i.e., flooding this year is independent of flooding in other years, and the probability remains constant from year to year).

Two outcomes, denoted as A and B, have the probability of A occurring equal to p and the probability of B occurring equal to (1 - p), which is denoted as q (i.e., q = 1 - p). If x is the number of occurrences of A, B occurs (n - x) times in n trials. One possible sequence of x occurrences of A and n - x occurrences of B would be:

$$A,A,A,...,A,B,B,...,B$$

Since the trials are independent, the probability of this sequence is the product of the probabilities of the n outcomes:

$$ppp \cdots p\,(1-p)(1-p)\,\cdots(1-p)$$

which is equal to:

$$p^x(1-p)^{n-x} = p^x q^{n-x} \tag{4.75}$$

There are many other possible sequences x occurrences of A and n - x occurrences of B, e.g.,

$$A,A,A,...,A,B,A,B,B,B,...,B$$

It would be easy to show that the probability of this sequence occurring is also given by Equation 4.75. In fact, any sequence involving x occurrences of A and (n - x) occurrences of B would have the probability given by Equation 4.75. Thus it is only necessary to determine how many different sequences of x occurrences of A and (n - x) occurrences of B are possible. It can be shown that the number of occurrences is:

$$\frac{n!}{x!(n-x)!} \tag{4.76}$$

where n! is read "n factorial" and equals:

$$n! = n(n-1)(n-2)\cdots(2)(1)$$

Computationally, the value of Equation 4.76 can be found from

$$\frac{n(n-1)\ \cdots\ (n-x+1)}{x!}$$

The quantity given by Equation 4.76 is computed so frequently that it is often abbreviated by $\binom{n}{x}$ and called the binomial coefficient. It represents the number of ways that sequences involving events A and B can occur with x occurrences of A and (n - x) occurrences of B. Combining Equations 4.76 and 4.77 gives the probability of getting exactly x occurrences of A in n trials, given that the probability of event A occurring on any trial is p:

$$b(x; n, p) = \binom{n}{x} p^{x} (1-p)^{n-x} \quad \text{for } x = 0, 1, 2, \ldots, n \tag{4.77}$$

This is a binomial probability, and the probabilities defined by Equation 4.76 represent the distribution of binomial probabilities. It is denoted as b(x; n, p), which is read "the probability of getting exactly x occurrences of a random variable in n trials when the probability of the event occurring on any one trial is p."

For example, if n equals 4 and x equals 2, Equation 4.76 would suggest six possible sequences:

$$\frac{4!}{2!(4-2)!} = \frac{(4)(3)(2)(1)}{(2)(1)(2)(1)} = 6 \tag{4.78}$$

The six possible sequences are (AABB), (ABBA), (ABAB), (BAAB), (BABA), and (BBAA). Thus if the probability of A occurring on any one trial is 0.3, then the probability of exactly two occurrences in four trials is:

$$b(2;4,0.3) = \binom{4}{2}(0.3)^{2}(1-0.3)^{4-2} = 0.2646$$

Similarly, if p equals 0.5, the probability of getting exactly two occurrences of event A would be

$$b(2;4,0.5) = \binom{4}{2}(0.5)^{2}(1-0.5)^{4-2} = 0.375$$

It is easy to show that for four trials there is only one way of getting either zero or four occurrences of A, there are four ways of getting either one or three occurrences of A, and there are six ways of getting two occurrences of A. Thus with a total of 16 possible outcomes, the value given by Equation 4.78 for the number of ways of getting two occurrences divided by the total of 16 possible outcomes supports the computed probability of 0.375.

Example 4.13. A coffer dam is to be built on a river bank so that a bridge pier can be built. The dam is designed to prevent flow from the river from interfering with the construction of the pier.

The cost of the dam is related to the height of the dam; as the height increases, the cost increases. But as the height is increased, the potential for flood damage decreases. The level of flow in the stream varies weekly and can be considered as a random variable. However, the design engineer is interested only in two states, the overtopping of the dam during a 1-workweek period or the non-overtopping. If construction of the pier is to require 2 years for completion, the time period consists of 104 independent "trials." If the probability of the flood that would cause overtopping remains constant (p), the problem satisfies the four assumptions required to use the binomial distribution for computing probabilities.

If x is defined as an occurrence of overtopping and the height of the dam is such that the probability of overtopping during any 1-week period is 0.05, then for a 104-week period (n = 104), the probability that the dam will not be overtopped (x = 0) is computed using Equation 4.77:

$$p(\text{ no overtopping }) = b(0; 104, 0.05) = \binom{104}{0}(0.05)^0 (0.95)^{104} = 0.0048$$

The probability of exactly one overtopping is

$$b(1; 104, 0.05) = \binom{104}{1}(0.05)^1 (0.95)^{103} = 0.0264$$

Thus the probability of more than one overtopping is:

$$1 - b(0; 104, 0.05) - b(1; 104, 0.05) = 0.9688$$

The probability of the dam not being overtopped can be increased by increasing the height of the dam. If the height of the dam is increased so that the probability of overtopping in a 1-week period is decreased to 0.02, the probability of no overtoppings increases to

$$p(\text{no overtoppings}) = b(0; 104, 0.02) = \binom{104}{0}(0.02)^0 (0.98)^{104} = 0.1223$$

Thus the probability of no overtopping during the 104-week period increased 25 times when the probability of overtopping during 1 week was decreased from 0.05 to 0.02.

4.6.2 Flood Risk

The probability of nonexceedence of Q_A given in Equation 4.4 can now be written in terms of the return period as:

$$P_r(\text{not } Q_A) = 1 - P_r(Q_A) = 1 - \frac{1}{T_r} \quad (4.79)$$

By expanding Equation 4.6, the probability that Q_A will not be exceeded for n successive years is given by:

$$P_r(\text{ not } Q_A)\, P_r(\text{ not } Q_A) \cdots P_r(\text{ not } Q_A) = [\, P_r(\text{ not } Q_A)]^n = \left[1 - \frac{1}{T_r}\right]^n \quad (4.80)$$

Risk, R, is defined as the probability that Q_1 will be exceeded at least once in n years:

$$R = 1 - [P_r(\text{not } Q_A)]^n = 1 - \left[1 - \frac{1}{T_r}\right]^n \qquad (4.81)$$

Equation 4.81 was used for the calculations of Table 4.25, which gives the risk of failure as a function of the project design life, n, and the design return period, T_r.

<u>Example 4.14</u>. The use of Equation 4.81 or Table 4.25 is illustrated by the following example. What is the risk that the 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 4.81, the risk is calculated as:

$$R = 1 - \left[1 - \frac{1}{T_r}\right]^n = 1 - \left[1 - \frac{1}{10}\right]^2 = 0.19$$

In other words, there is about a 20 percent chance that this structure will be subjected to a 10-year flood in the first 2 years of its life.

Table 4.25. Risk of Failure(R) as a Function of Project Life (n) and Return Period (T_r)

	Return Period (T_r)					
n	**2**	**5**	**10**	**25**	**50**	**100**
1	0.500	0.200	0.100	0.040	0.020	0.010
3	0.875	0.488	0.271	0.115	0.059	0.030
5	0.969	0.672	0.410	0.185	0.096	0.049
10	0.999	0.893	0.651	0.335	0.183	0.096
20		0.988	0.878	0.558	0.332	0.182
50			0.995	0.870	0.636	0.395
100				0.983	0.867	0.634

CHAPTER 5

PEAK FLOW FOR UNGAGED SITES

While using frequency approaches is almost always the most appropriate means to determine a peak flow, 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 making a flood frequency analysis, such as a log-Pearson Type III analysis. Several regional analysis and empirical techniques have been developed and successfully applied to address these situations.

Extrapolation of data from nearby watersheds with comparable hydrologic and physiographic features is referred to as regional analysis and includes regional regression equations and index-flood methods. The USGS has collected a comprehensive series of these regional regression equations into the National Flood Frequency computer program. This tool provides the means for computing a peak discharge for any place in the United States.

Empirical methods include such widely applied techniques as the rational formula and the NRCS (formerly the SCS) graphical method. These methods employ empirical relationships between rainfall and runoff that allow estimation of design discharges on ungaged watersheds by development of parameters describing the watershed. If an engineer has an interest in the magnitude of measured maximum flood flows, peak discharge envelope curves can be used alone or in conjunction with other regional or empirical analyses.

Watershed area plays an important role for each of these ungaged watershed peak flow determination methods. As described in Chapter 2, watershed area is the single most important characteristic for determining runoff peaks. As will be seen, the area of the watershed also provides a basis for determining the limits of applicability for many of these methods.

5.1 REGIONAL REGRESSION EQUATIONS

Regional regression equations are commonly used for estimating peak flows at ungaged sites or sites with insufficient data. Regional regression equations 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.

5.1.1 Analysis Procedure

The typical multiple regression model utilized in regional flood studies uses the power model structure:

$$Y_T = aX_1^{b_1} X_2^{b_2} \cdots X_p^{b_p} \qquad (5.1)$$

where,

Y_t = the dependent variable
$X_1, X_2, ..., X_p$ = independent variables
a = the intercept coefficient
$b_1, b_2, ..., b_p$ = regression coefficients.

The dependent variable is usually the peak flow for a given return period T 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, $b_1, b_2, ..., b_p$ are determined using a regression analysis. Regression analysis is described in detail by Sanders

(1980), Riggs (1968), and McCuen (1993). The general procedure for making a regional regression analysis is as follows:

1. Obtain the annual maximum flood series for each of the gaged sites in the region.

2. Perform a separate flood frequency analysis (e.g., log-Pearson Type III) on each of the flood series of Step 1 and determine the peak discharges for selected return periods (e.g., the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year discharges are commonly selected).

3. Determine the values of watershed and meteorological characteristics for each watershed for which a flood series was collected in Step 1.

4. Form an (n by p) data matrix of all the data collected in Step 3, where n is the number of watersheds of step 1 and p is the number of watershed characteristics obtained for Step 3.

5. Form a one-dimensional vector with n peak discharges for the specific return period selected.

6. Regress the vector of n peak discharges of Step 5 on the data matrix of Step 4 to obtain the prediction equation.

If more than one return period is of interest, the procedure can be repeated for each return period, with a separate equation developed for each return period. In this case, it is also important to review closely the regression coefficients to ensure that they are rational and consistent across the various return periods. Because of sampling variation, it is possible for the regression analyses to produce a set of coefficients that, under certain sets of values for the predictor variables, result in the computed 10-year discharge, for example, being greater than the computed 25-year discharge. In such cases, the irrational predictions can be eliminated by smoothing the coefficients. If the coefficients need to be smoothed, the goodness-of-fit statistics should be recomputed using the smoothed coefficients. The problem can usually be prevented by using the same predictor variables for all of the equations.

The most important watershed characteristic is usually 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 slope, length, and geometry, shape factors, watershed perimeter, aspect, elevation, basin fall, land use, and others. Meteorological characteristics that are 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 level of significance (e.g., 5 percent) can be eliminated. In addition to statistical criteria, it is also important for all coefficients to be reasonable.

The specific predictor variables to be included in a regression equation are usually selected using a stepwise regression analysis (McCuen, 1989). While a 5 percent level of significance is sometimes used to make the decision, it is better to select only those variables that are easily obtained and necessary to provide both a reasonable level of accuracy and rational coefficients. When stepwise regression analysis is used to select variables for a set of equations for different return periods, the same independent variables should be used in all of the equations. In a few cases, this may cause some equations in the set to have less accuracy than would be possible, but it is usually necessary to ensure consistency across the set of equations.

5.1.2 USGS Regression Equations

In a series of studies by the USGS, the Federal Highway Administration, and State Highway Departments, statewide regression equations have now been developed throughout the United States. The highway community has made a significant contribution to acquiring additional stream flow data through funding USGS stream gaging station studies throughout the country since the 1960s. Highway interests have supported these research endeavors with expenditures of $14 million. These equations permit peak flows to be estimated for return periods varying from 2 to 500 years. The published equations (Jennings, et al., 1994) are included in the National Flood Frequency (NFF) Program discussed in Section 5.1.5.2.

Typically, each state is divided into regions of similar hydrologic, meteorologic, and physiographic characteristics as determined by various hydrological and statistical measures. 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 regression was then used on the logarithmically 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 a predetermined level of significance were retained in the final equations.

5.1.2.1 Hydrologic Flood Regions

In most statewide flood-frequency reports, the analysts divided the state into separate hydrologic regions. Regions of homogeneous flood characteristics were generally determined by using major watershed boundaries and an analysis of the areal distribution of the regression residuals, which are the differences between regression and station (observed) T-year estimates. In some instances, the hydrologic regions were also defined by the mean elevation of the watershed or by statistical tests such as the Wilcoxon signed-rank test.

Regression equations are defined for 210 hydrologic regions throughout the Nation, indicating that, on average, there are about four regions per state. Figure 5.1 gives the NFF statewide results for Maine and is used to illustrate the content for one of the 210 regions. Some areas of the Nation, however, have inadequate data to define flood-frequency regions. For example, there are regions of undefined flood frequency in Florida, Texas, and Nevada. For the state of Hawaii, regression equations are only provided for the island of Oahu.

Summary

Maine is considered to be a single hydrologic region. The regression equations developed for the state are for estimating peak discharges (QT) having recurrence intervals T that range from 2 to 100 years. The explanatory basin variables used in the equations are drainage area (A), in square miles; channel slope (S), in feet per mile; and storage (St), which is the area of lakes and ponds in the basin in percentage of total area. The constant 1 is added to St in the computer application of the regression equations. The user should enter the actual value of St. All variables can be measured from topographic maps. The regression equations were developed from peak-discharge records through 1974 for 60 sites with records of at least 10 years in length. The regression equations apply to streams having drainage areas greater than 1 square mile and virtually natural flood flows. Standard errors of estimate of the regression equations range from 31 to 49 percent.

Procedure

Topographic maps and the following equations are used to estimate the needed peak discharges QT, in cubic feet per second, having selected recurrence intervals T.

$$Q2 = 14.0A^{0.962}S^{0.268}ST^{-0.212}$$

$$Q5 = 21.2A^{0.946}S^{0.298}ST^{-0.239}$$

$$Q10 = 26.9A^{0.936}S^{0.315}ST^{-0.252}$$

$$Q25 = 35.6A^{0.923}S^{0.333}ST^{-0.266}$$

$$Q50 = 42.7A^{0.915}S^{0.346}ST^{-0.275}$$

$$Q100 = 50.9A^{0.907}S^{0.358}ST^{-0.282}$$

Reference

Morrill, R.A., 1975. “A Technique for Estimating the Magnitude and Frequency of Floods in Maine.” U.S. Geological Survey Open-File Report No. 75-292.

Figure 5.1. Description of NFF regression equations for rural watersheds in Maine (Jennings, et al., 1994).

<u>Example 5.1</u>. 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 (1970). The equations have the following form:

$$Q_T = a\,A^{b_1}\,S^{b_2} \qquad (5.2)$$

where,

Q_T = peak annual flow for the specified return periods, m^3/s (ft^3/s)
A = drainage area contributing surface runoff above the site, km^2 (mi^2)
S = 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, m/km (ft/mi).

The coefficients of Equation 5.2 are given in Table 5.1. The range of application of the above equations was specified as:

Variable	Value in SI	Value in CU
Drainage Area (A)	$2.80 < A\ (km^2) < 5{,}040$	$1.08 < A\ (mi^2) < 1{,}950$
Slope (S)	$1.7 < S\ (m/km) < 14.5$	$9.2 < S\ (ft/mi) < 76.8$

By measuring the drainage area above the site from a topographic map, the area A is found to be 545.5 km^2 (210.6 mi^2) and the channel slope between the 10 and 85 percent points is 2.833 m/km (14.96 ft/mi). Using Equation 5.2 and the coefficients of Table 5.1, the 25-year peak flow is:

Variable	Value in SI	Value in CU
$Q_{25} = a_{25}\ A^{0.776}\ S^{0.554}$	$= 6.13(545.5)^{0.776}(2.833)^{0.554}$ $= 1450\ m^3/s$	$= 180(210.6)^{0.776}\ (14.96)^{0.554}$ $= 51{,}200\ ft^3/s$

Table 5.1. Regression Coefficients for Texas, Region 5

Return Period, T (years)	Regression Coefficients				Standard Error (%)*
	a (SI)	a (CU)	b_1	b_2	
2	0.319	4.82	0.799	0.966	62.1
5	1.60	36.4	0.776	0.706	46.6
10	3.15	82.6	0.776	0.622	42.6
25	6.13	180	0.776	0.554	41.3
50	8.96	278	0.778	0.522	42.0
100	12.3	399	0.782	0.497	44.1

* Standard errors were computed using the logarithmic regression and are given as a percentage of the mean.

5.1.2.2 Assessing Prediction Accuracy

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 estimate is a measure of the deviation of the observed data from the corresponding predicted values and is given by the basic equation:

$$S_e = \left[\frac{\Sigma(\hat{Q}_i - Q_i)^2}{n - q} \right]^{0.5} \quad (5.3)$$

where,

Q_i = observed value of the dependent variable (discharge)

$\hat{Q}_i$ = corresponding value predicted by the regression equation

n = number of watersheds used in developing the regression equation

q = number of regression coefficients (i.e., a, b_1, ..., b_p).

In a manner analogous to the variance, the standard error can be expressed as a percentage by dividing the standard error S_e by the mean value ($\overline{Q}_T$) of the dependent variable:

$$V_e = \frac{S_e}{\overline{\overline{Q}}_T} \times 100\% \quad (5.4)$$

where,

V_e = coefficient of error variation.

V_e of Equation 5.4 has the form of the coefficient of variation of Equation 4.14. The standard error of regression S_e has a very similar meaning to that of the standard deviation, Equation 4.13, for a normal distribution in that approximately 68 percent of the observed data should be contained within ±1 standard error of the regression line.

When S_e is computed for regional regression equations, it is usually computed using the logarithms of the flows. Thus, $\hat{Q}_i$ and Q_i of Equation 5.3 are logarithms of the corresponding flows. This is believed to be necessary because the errors (i.e., $\hat{Q}_i - Q_i$) have a constant variance when expressed from the logarithms.

5.1.2.3 Comparison with Gaged Estimates

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 Chapter 4. A direct comparison cannot be made with the previously used Medina River data because of 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 1,228 km^2 (474 mi^2), is totally unregulated, and has station and generalized skews of -0.005 and -0.234, respectively. The data were analyzed with a log-Pearson Type III distribution, and the 10-, 25-, 50- and 100-year peak discharges estimated using the USGS Bulletin 17B (1982) weighted skew option (G_L = -0.2). These values together with peak flows determined from a frequency curve through the systematic record are summarized in Table 5.2.

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 3.07 m/km (16.2 ft/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 5.2.

The peak discharges estimated from the regression equations are all substantially higher than the comparable values determined from the log-Pearson Type III analysis, although all are within the USGS 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 Type 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 7960 m^3/s (281,000 ft^3/s), which, according to the log-Pearson Type 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 the gage. Some justification exists for this argument as there are many examples throughout the country of adjacent watersheds of comparable size and physiographic and hydrologic characteristics experiencing the same storm patterns, but 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.

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 also provide insight into the treatment of outliers beyond the purely statistical methods discussed in Section 4.3.6.1. As demonstrated by the above discussion, comparison of the peak flows obtained by different methods may 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.

Sauer (1973) has proposed a methodology for weighting the log-Pearson Type III result with the regression equation estimate for the gaged watershed based on the gage record length and the equivalent record length for the regression equation as follows:

$$Q_{gw} = \frac{Q_g N_g + Q_r N_r}{N_g + N_r} \qquad (5.5)$$

where,

Q_{gw} = weighted peak flow estimate at the gage
Q_g = log-Pearson Type III peak flow estimate at the gage
Q_r = regression equation peak flow estimate at the gage
N_g = number of years of record at the gage
N_r = equivalent record length of the regression equation.

This methodology seeks to use information in the gage record as well as similar gaged watersheds in the region via the regression equations. It is presented in many of the USGS reports documenting development of the regression equations.

Table 5.2. Comparison of Peak Flows from Log-Pearson Type III Distribution and USGS Regional Regression Equation

	Peak Discharge (m^3/s)			Peak Discharge (ft^3/s)		
Return Period (years)	Log-Pearson Type III Frequency	Systematic Record	USGS Regression Equations	Log-Pearson Type III Frequency	Systematic Record	USGS Regression Equations
10	1,210	1,420	1,580	42,700	50,300	55,700
25	1,950	2,520	2,850	68,900	89,000	100,000
50	2,630	3,640	4,070	92,900	129,000	144,000
100	3,420	5,080	5,590	120,900	179,000	197,000

5.1.2.4 Application and Limitations

Several points 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. This includes applying the equations to basins that fall within the range of characteristics for basins used to develop the equations. The designer is also cautioned to apply these equations within or close to 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 reconciled.

The following additional limitations should be observed:

- Rural equations should only be used for rural areas and should not be used in urban areas unless the effects of urbanization are insignificant.

- Regression equations should not be used where dams, flood-detention structures, and other human-made works have a significant effect on peak discharges.

- The magnitude of the standard errors can be larger than the reported errors if the equations are used to estimate flood magnitudes for streams with variables outside the ranges for the necessary input variables as stated in the applicable report.

- Drainage area should always be determined. Although a hydrologic region might not include drainage area as a variable in the prediction equation to compute a frequency curve, the drainage area may be used for determining the maximum flood envelope discharge from Crippen and Bue (1977) and Crippen (1982), as well as weighting of curves for watersheds in more than one region.

- Frequency curves for watersheds contained in more than one region cannot be computed if the regions involved do not have corresponding T-year equations. Failure to observe this limitation will lead to erroneous results. Frequency curves are weighted by the percentage of drainage area in each region. No provision is provided for weighting frequency curves for watersheds in two different states.

- In some instances, the maximum flood envelope value might be less than some T-year computed peak discharges for a given watershed. The T-year peak discharge is the discharge that will be exceeded as an annual maximum peak discharge, on average, once every T years. The engineer should carefully determine which maximum flood-region contains the watershed being analyzed and is encouraged to consult Crippen and Bue (1977) and Crippen (1982) for guidance and interpretations.

- The engineer should be cautioned that some hydrologic regions do not have prediction equations for peak discharges as large as the 100-year peak discharge. The engineer is responsible for the assessment and interpretation of any interpolated or any extrapolated T-year peak discharge. Examination of plots of the frequency curves is highly desirable.

Maximum flood envelopes are discussed later in this chapter.

5.1.3 USGS Urban Watershed Studies

In 1978, the Federal Highway Administration contracted with the USGS 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 (Sauer, et al., 1983).

A review of nearly 600 urbanized sites resulted in a final list of 269 sites that met criteria wherein at least 15 percent of the drainage area was covered with commercial, industrial, or residential development; reliable flood 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 record was coincident with a period of relatively constant urbanization. The complete data base, including topographic and climatic variables, land use variables, urbanization indices, and flood frequency estimates are available from 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 relate the peak discharge at a specified frequency to: (l) 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 that exists (or might exist in the future) in the watershed. The BDF is discussed in more detail in Section 5.1.4.

The USGS regression equations can be used to estimate the peak discharge for existing conditions of urbanization, and they can also be used to estimate the peak discharge for future conditions. 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. However, these estimates cannot be treated as absolutes, and some judgment must be exercised in their application.

5.1.3.1 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 increasing the standard error of regression. Ultimately, the following family of three-parameter equations was developed by the USGS for peak discharges in urbanized watersheds:

$$UQ_T = a_T \; A^{C_{1T}} (13 - BDF)^{C_{2T}} \; RQ_T^{C_{3T}} \qquad (5.6)$$

where,

UQ_T = peak discharge of recurrence interval, T, for an urbanized condition, m^3/s (ft^3/s)
T = recurrence interval ranging from 2 to 500 years
A = drainage area of the basin, km^2 (mi^2)
BDF = basin development factor as defined below
RQ_T = peak discharge of recurrence interval, T, for rural conditions, m^3/s (ft^3/s).
A_T, C_{1T}, C_{2T}, and C_{3T} = regression constants summarized in Table 5.3.

This equation is applicable for watersheds between 0.5 and 260 km^2 (0.2 and 100 mi^2).

Table 5.3. Unit Conversion Constants for the USGS Urban Equations

Return Period	a_T (SI)	a_T (CU)	C_{1T}	C_{2T}	C_{3T}
2	4.13	13.2	0.21	-0.43	0.73
5	4.12	10.6	0.17	-0.39	0.78
10	3.86	9.51	0.16	-0.36	0.79
25	3.69	8.68	0.15	-0.34	0.80
50	3.54	8.04	0.15	-0.32	0.81
100	3.52	7.70	0.15	-0.32	0.82
500	3.38	7.47	0.16	-0.30	0.82

5.1.3.2 Basin Development Factor

Several indices of urbanization were evaluated in the course of the USGS study including percentage of basin occupied by impervious surfaces, population and population density, basin response time, and basin development factor. The BDF, which provides a measure of the efficiency of the drainage system within an urbanizing watershed, was selected for a number of reasons. The BDF was highly significant in the regression equations, compared to the other measures of urbanization, and its value may be determined from topographic maps, storm drain maps, and field surveys.

To determine the BDF, the basin is first divided into three sections as shown in Figure 5.2. Each section contains approximately one-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:

1. **Channel modifications**. If channel modifications 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), a code of 1 is assigned. Any one, or all, of these modifications would qualify for a code of 1. To be considered significant, at least 50 percent of the main drainage channels and principal tributaries must be modified to some extent over natural conditions. If channel modifications are not prevalent, a code of 0 is assigned.

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, a code of 1 is assigned. If less than 50 percent of these channels are lined, a code of 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 channels that 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 consist of storm drains, a code of 1 is assigned. If less than 50 percent of the secondary tributaries consist of storm drains, a code of 0 is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a code of 1.

4. **Urbanization/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, a code of 1 should be assigned. Otherwise, a code of 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. Practical determination involves a certain amount of subjectivity and engineering judgment. It is recommended that field checking be performed to obtain the best estimate. The BDF is computed as the sum of the assigned codes. With three subareas per basin, and four drainage aspects to which codes are assigned in each subarea, the maximum

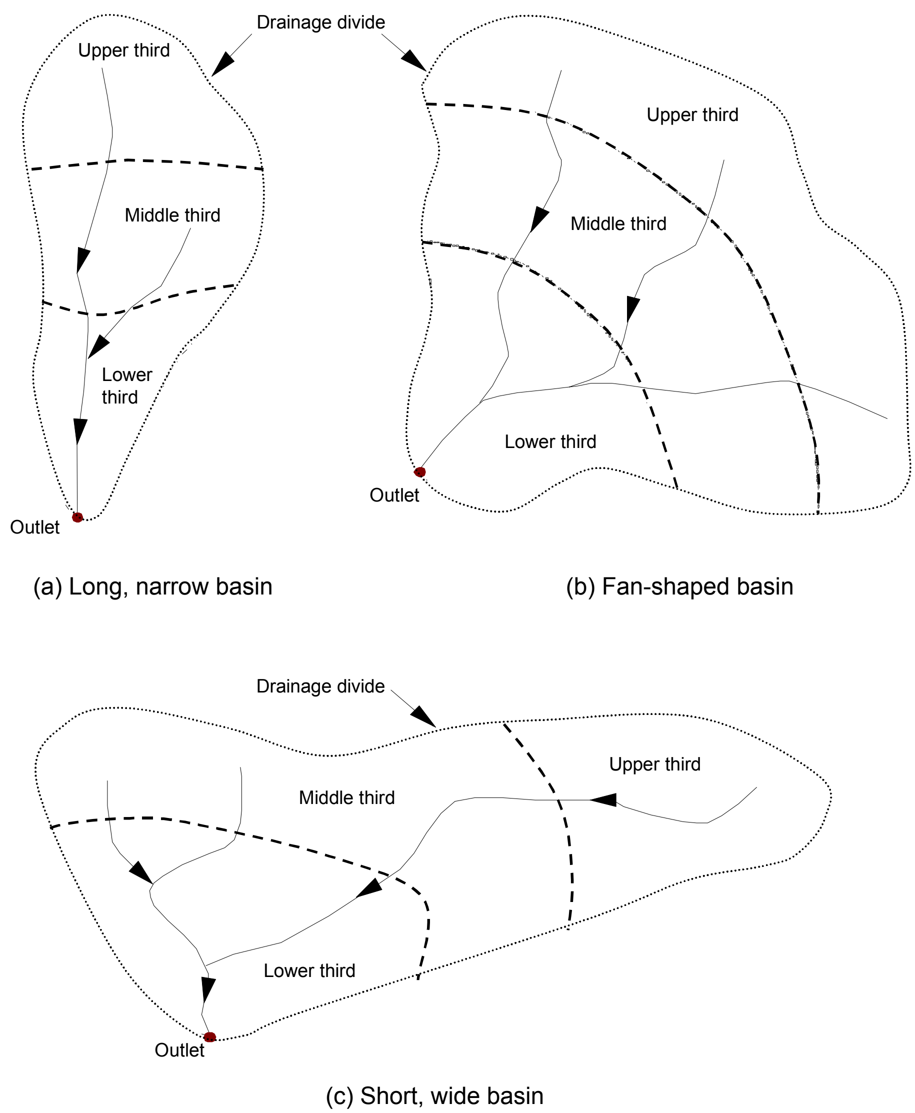


Figure 5.2. Subdivision of watersheds for determination of BDF

value for a fully developed drainage system would be 12. Conversely, if the drainage system has not been developed, a BDF of 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 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. 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 development plans.

Example 5.2 (SI). Information is first collected from topographic maps and a field survey for the 99.3-ha watershed. The watershed is divided into three subareas of approximately equal area. The separation is based on homogeneity of hydrologic conditions, with the following values measured:

Subarea	**Area (ha)**	**Main Channel Length (m)**	**Length of Secondary Tributaries (m)**	**Road Length (m)**	**Length of Channel Modified (m)**	**Length of Channel Lined (m)**	**Storm Drains (m)**	**Curb and Gutter (m)**
Upper	29.2	780	1,580	870	140	0	410	210
Middle	36.3	1,140	1,200	1,430	615	540	680	920
Lower	33.8	910	660	1,710	525	480	460	970
Sum	99.3	2,830						

The BDF is determined as follows:

Channel Modifications
Upper Third: 140 m have been straightened and deepened — Code = 0
[140/780 < 50%]
Middle Third: 615 m have been straightened and deepened — = 1
[615/1140 > 50%]
Lower Third: 525 m have been straightened and widened — = 1
[525/910 > 50%]

Channel Linings
Upper Third: 0 m of channel are lined — Code = 0
[0/780 < 50%]
Middle Third: 540 m of channel are lined — = 0
[540/1140 < 50%]
Lower Third: 480 m of channel are lined — = 1
[480/910 > 50%]

Storm Drains on Secondary Tributaries
Upper Third: 410 m have been converted to drains — Code = 0
[410/1580 < 50%]
Middle Third: 680 m have been converted to drains — = 1
[680/1200 > 50%]
Lower Third: 460 m have been converted to drains — = 1
[460/660 > 50%]

Curb and Gutter Streets
Upper Third: 20% urbanized with 210 m curb/gutter — Code = 0
[210/870 < 50%]
Middle Third: 70% urbanized with 920 m curb/gutter — = 1
[920/1430 > 50%]
Lower Third: 55% urbanized with 970 m curb/gutter — = 1
[970/1710 > 50%]

Total BDF = 7

Example 5.2 (CU). Information is first collected from topographic maps and a field survey for the following watershed. The watershed is divided into three subareas of approximately equal area. The separation is based on homogeneity of hydrologic conditions, with the following values measured:

Subarea	**Area (ac)**	**Main Channel Length (ft)**	**Length of Secondary Tributaries (ft)**	**Road Length (ft)**	**Length of Channel Modified (ft)**	**Length of Channel Lined (ft)**	**Storm Drains (ft)**	**Curb and Gutter (ft)**
Upper	72.2	2,560	5,180	2,850	460	0	1,350	690
Middle	89.7	3,740	3,940	4,690	2,020	1,770	2,230	3,020
Lower	83.5	2,990	2,170	5,610	1,720	1,570	1,510	3,180
Sum	245.4	9,290						

The BDF is determined as follows:

Channel Modifications

Upper Third: 460 ft have been straightened and deepened [460/2,560 < 50%]	Code	= 0
Middle Third: 2,020 ft have been straightened and deepened [2,020/3,740 > 50%]		= 1
Lower Third: 1,720 have been straightened and widened [1,720/2,990 > 50%]		= 1

Channel Linings

Upper Third: 0 ft of channel are lined [0/2,560 < 50%]	Code	= 0
Middle Third: 1,770 ft of channel are lined [1,770/3,740 < 50%]		= 0
Lower Third: 1,570 of channel are lined [1,570/2,990 > 50%]		= 1

Storm Drains on Secondary Tributaries

Upper Third: 1,350 ft have been converted to drains [1,350/5,180 < 50%]	Code	= 0
Middle Third: 2,230 ft have been converted to drains [2,230/3,940 > 50%]		= 1
Lower Third: 1,510 ft have been converted to drains [1,510/2,170 > 50%]		= 1

Curb and Gutter Streets

Upper Third: 20% urbanized with 690 ft curb/gutter [690/2,850 < 50%]	Code	= 0
Middle Third: 70% urbanized with 3,020 ft curb/gutter [3,020/4,690 > 50%]		= 1
Lower Third: 55% urbanized with 3,180 ft curb/gutter [3,180/5,610 > 50%]		= 1
	Total BDF	= 7

Example 5.3. The 25-year peak discharge is computed for an urban watershed of 67 km^2 (26 mi^2) with a BDF of 4. The percentage increase over the undeveloped rural condition is also computed.

1. Determine the equivalent rural discharge using the published USGS statewide regression equation. For this site, the 25-year peak discharge for the rural conditions is determined from the following equation:

Variable	Value in SI	Value in CU
$RQ_{25} = a_{25} A^{C_T}$	$= 4.21(67)^{0.666} = 69\ m^3/s$	$= 280(26)^{0.666} = 2450\ ft^3/s$

2. Determine the urbanized discharge:

$$UQ_{25} = a_{25}\ A^{C_{1,25}} (13 - BDF)^{C_{2,25}}\ RQ_{25}^{C_{3,25}}$$

Value in SI	Value in CU
$UQ_{25} = 3.69 A^{0.15} (13 - BDF)^{-0.34} RQ^{0.80}$ $= 3.69(67)^{0.15}(13-4)^{-0.34}(69)^{0.80}$ $= 97\ m^3/s$	$UQ_{25} = 8.68 A^{0.15} (13 - BDF)^{-0.34} RQ^{0.80}$ $= 8.68(26)^{0.15}(13-4)^{-0.34}(2{,}450)^{0.80}$ $= 3{,}450\ ft^3/s$

3. Determine the percent change:

Variable	Value in SI	Value in CU
$\%\ change = \frac{UQ_{25} - RQ_{25}}{RQ_{25}} \times 100\%$	$= \frac{97 - 69}{69} \times 100\% = 41\%$	$= \frac{3450 - 2450}{2450} \times 100\% = 41\%$

5.1.3.3 Effects of Future Urbanization

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. This is illustrated by showing the impact on the 5-year peak discharge when the BDF changes from 5 to 10.

For the present and future conditions, the 5-yr peak discharge is computed with Equation 5.6:

$$UQ_5 = a_5\ A^{0.17} (13 - BDF_i)^{-0.39}\ RQ_5^{0.78}$$

where i = p and i = f for the present and the future BDF, respectively. The change in the BDF is:

$$\Delta BDF = (\ BDF_f - BDF_p\ ) \qquad (5.7)$$

which can be rearranged to:

$$BDF_f = BDF_p + \Delta BDF \tag{5.8}$$

The ratio of the future UQ_{5f} to the present UQ_{5p} is:

$$\frac{UQ_{5f}}{UQ_{5p}} = \frac{a_5 \ A^{0.17} \ \left[13 - \left(BDF_p + \Delta BDF\right)\right]^{-0.39} RQ_5^{0.78}}{a_5 \ A^{0.17} \ (13 - BDF_p)^{0.39} RQ_5^{0.78}} \tag{5.9}$$

Canceling the common terms and rearranging yields:

$$\frac{UQ_{5f}}{UQ_{5p}} = \left[1 - \frac{\Delta BDF}{13 - BDF_p}\right]^{-0.39} \tag{5.10}$$

For this example, $BDF_p = 5$ and $\Delta BDF = (10 - 5)$; therefore:

$$\frac{UQ_{5f}}{UQ_{5p}} = \left[1 - \frac{5}{8}\right]^{-0.39} = 1.47$$

Thus, 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:

$$\frac{UQ_f}{UQ_p} = \left[1 - \frac{\Delta BDF}{13 - BDF_p}\right]^{C_{2T}} \tag{5.11}$$

where C_{2T} varies with recurrence intervals as given in Table 5.3.

5.1.3.4 Local Urban Equations

Many of the USGS regression studies include additional equations for some cities and metropolitan areas that were developed for local use in those designated areas only. These local urban equations can be used in lieu of the nationwide urban equations, or they can be used for comparative purposes. It would be highly coincidental for the local equations and the nationwide equations to give identical results.

Therefore, it is advisable to compare results of the two (or more) sets of urban equations, and to also compare the urban results to the equivalent rural results. Ultimately, it is the engineer's decision as to which urban results to use.

Local urban equations are available in many cities throughout the United States. In addition, some of the rural reports contain estimation techniques for urban watersheds. Several of the rural reports suggest the use of the nationwide equations given by Sauer, et al. (1983).

5.1.4 National Flood Frequency Program

Because of the common usage of the USGS equations developed for individual states and regions, the USGS has developed software called the National Flood Frequency Program (Jennings, Thomas, and Riggs, 1994). The USGS, in cooperation with the Federal Highway Administration and the Federal Emergency Management Agency, has compiled all of the current (as of September 1993) statewide and metropolitan area regression equations into a microcomputer program titled the National Flood Frequency Program. This program summarizes regression equations for estimating flood-peak discharges and techniques for estimating a typical flood hydrograph for a given recurrence interval or exceedence probability peak discharge for unregulated rural and urban watersheds. The report summarizes the statewide regression equations for rural watersheds in each state, summarizes the applicable metropolitan area or tatewide regression equations for urban watersheds, describes the National Flood Frequency software for making these computations, and provides much of the reference information and input data needed to run the computer program.

Since 1973, regression equations for estimating flood-peak discharges for rural, unregulated watersheds have been published, at least once, for every state and the Commonwealth of Puerto Rico. For some areas of the Nation, however, data are still inadequate to define flood-frequency characteristics. Regression equations for estimating urban flood-peak discharges for many metropolitan areas are also available. Typical flood hydrographs corresponding to a given rural and urban peak discharge can also be estimated by procedures described in the NFF report.

Information on computer specifications and the computer program is presented in appendices of the NFF report. Instructions for installing NFF on a personal computer are also given, in addition to a description of the NFF program and the associated database of regression statistics.

5.1.5 FHWA Regression Equations

In 1977, the Federal Highway Administration published a two-volume report by Fletcher, et al. (1977) that presents nationwide regression equations for predicting runoff from small rural watersheds (<130 km^2 or <50 mi^2). This method is not the equivalent of the USGS regression equations. While it was used rather widely at first, it is rarely used today. The procedure is similar in concept to that of Potter (1961). It was developed using frequency analyses 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 m^3/s (ft^3/s), Fletcher, et al. (1977) give the equation:

$$Q_{p(max)} = 10^{[C_0 + C_1 \log A + C_2 (\log A)^2]} \tag{5.12}$$

where,

log A = base-10 logarithm of the drainage area, km^2 (mi^2)

$Q_{p(max)}$ = discharge, m^3/s (ft^3/s)

C_0 , C_1 , and C_2 = regression coefficients equal to 2.031, 0.8389, and –0.0325, respectively, in SI units and 3.920, 0.8120, and –0.0325, respectively, in CU units.

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:

$$\hat{q}_{10} = a\, A^{b_1} R^{b_2} E_c^{b_3} \tag{5.13}$$

where,

$\hat{q}_{10}$ = 10-year peak discharge, m^3/s (ft^3/s)

A = drainage area, km^2 (mi^2)

R = isoerodent factor defined as the product of the mean annual rainfall kinetic energy and the maximum respective 30-minute annual maximum rainfall intensity

E_c = difference in elevation measured along the main channel from the drainage structure site to the drainage basin boundary, m (ft)

a, b_1, b_2, and b_3 = regression coefficients.

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. (1977).

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 5.13 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 q_{10}. The second option involves the use of an all-zone equation developed from all of the data. The all-zone, three-parameter equation for the 10-year peak discharge, $q_{10(3AZ)}$, is:

$$\hat{q}_{10(3AZ)} = 0.02598 \; A^{0.56172} \; R^{0.94356} \; E_c^{0.16887} \tag{5.14}$$

For each of the 24 hydrophysiographic zones, is a correction equation is presented to adjust Equation 5.15 for zonal bias. These correction equations have the form:

$$\hat{q}_{10} = a_1 \; \hat{q}_{10(3AZ)}^{b_1} \tag{5.15}$$

where,

a_1 and b_1 = regression coefficients.

If the surface area of surface water storage is more than about 4 percent of the total drainage area, it is recommended that the value of 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. (1977) presented the following equations from which a frequency curve can be drawn on any appropriate probability paper:

$$Q_{2.33} = 0.47329\ \hat{q}_{10}^{1.00243} \quad (5.16)$$

$$Q_{50} = 1.58666\ \hat{q}_{10}^{1.02342} \quad (5.17)$$

$$Q_{100} = 1.82393\ \hat{q}_{10}^{1.02918} \quad (5.18)$$

where,

$Q_{2.33}$ = mean annual peak flow taken at a return period of 2.33 years

Q_{50} and Q_{100} = 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. (1977) 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 SCS GRAPHICAL PEAK DISCHARGE METHOD

For many peak discharge estimation methods, the input includes variables to reflect the size of the contributing area, the amount of rainfall, the potential watershed storage, and the time-area distribution of the watershed. These are often translated into input variables such as the drainage area, the depth of rainfall, an index reflecting land use and soil type, and the time of concentration. The SCS graphical peak discharge method is typical of many peak discharge methods that are based on input such as that described.

5.2.1 Runoff Depth Estimation

The volume of storm runoff can depend on a number of factors. Certainly, the volume of rainfall will be an important factor. For very large watersheds, the volume of runoff from one storm event may depend on rainfall that occurred during previous storm events. However, when using the design storm approach, the assumption of storm independence is quite common.

In addition to rainfall, other factors affect the volume of runoff. A common assumption in hydrologic modeling is that the rainfall available for runoff is separated into three parts: direct (or storm) runoff, initial abstraction, and losses. Factors that affect the split between losses and direct runoff include the volume of rainfall, land cover and use, soil type, and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage.

In developing the SCS rainfall-runoff relationship, the total rainfall was separated into three components: direct runoff (Q), actual retention (F), and the initial abstraction (I_a). The retention F was assumed to be a function of the depths of rainfall and runoff and the initial abstraction. The development of the equation yielded:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{5.19}$$

where,

P = depth of precipitation, mm (in)
I_a = initial abstraction, mm (in)
S = maximum potential retention, mm (in)
Q= depth of direct runoff, mm (in).

Given Equation 5.19, two unknowns need to be estimated, S and I_a. The retention S should be a function of the following five factors: land use, interception, infiltration, depression storage, and antecedent moisture.

Empirical evidence resulted in the following equation for estimating the initial abstraction:

$$I_a = 0.2S \tag{5.20}$$

If the five factors above affect S, they also affect I_a. Substituting Equation 5.20 into Equation 5.19 yields the following equation, which contains the single unknown S:

$$Q = \frac{(P - 0.2\ S)^2}{P + 0.8\ S} \tag{5.21}$$

Equation 5.21 represents the basic equation for computing the runoff depth, Q, for a given rainfall depth, P. It is worthwhile noting that while Q and P have units of depth, Q and P reflect volumes and are often referred to as volumes because it is usually assumed that the same depths occurred over the entire watershed.

Additional empirical analyses were made to estimate the value of S. The studies found that S was related to soil type, land cover, and the hydrologic condition of the watershed. These are represented by the runoff curve number (CN), which is used to estimate S by:

$$S = \alpha \left[\frac{1000}{CN} - 10 \right] \tag{5.22}$$

where

CN = index that represents the combination of a hydrologic soil group and a land use and treatment class

α = unit conversion constant equal to 25.4 in SI units and 1.0 in CU units.

Empirical analyses suggested that the CN was a function of three factors: soil group, the cover complex, and antecedent moisture conditions.

5.2.2 Soil Group Classification

SCS developed a soil classification system that consists of four groups, which are identified by the letters A, B, C, and D. Soil characteristics that are associated with each group are as follows:

Group A: deep sand, deep loess; aggregated silts

Group B: shallow loess; sandy loam

Group C: clay loams; shallow sandy loam; soils low in organic content; soils usually high in clay

Group D: soils that swell significantly when wet; heavy plastic clays; certain saline soils

The SCS soil group can be identified at a site using either soil characteristics or county soil surveys. The soil characteristics associated with each group are listed above and provide one means of identifying the SCS soil group. County soil surveys, which are made available by Soil Conservation Districts, give detailed descriptions of the soils at locations within a county; these surveys are usually the better means of identifying the soil group. Many of the more recent reports actually categorize the soils into these four groups.

5.2.3 Cover Complex Classification

The SCS cover complex classification consists of three factors: land use, treatment or practice, and hydrologic condition. Many different land uses are identified in the tables for estimating runoff curve numbers. Agricultural land uses are often subdivided by treatment or practices, such as contoured or straight row; this separation reflects the different hydrologic runoff potential that is associated with variation in land treatment. The hydrologic condition reflects the level of land management; it is separated into three classes: poor, fair, and good. Not all of the land uses are separated by treatment or condition.

5.2.4 Curve Number Tables

Table 5.4 shows the SCS CN values for the different land uses, treatments, and hydrologic conditions; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, the CN will be 66.

Table 5.4. Runoff Curve Numbers
(average watershed condition, $I_a = 0.2S$)(After: SCS, 1986)

Cover Type		Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Fully developed urban areas[a] (vegetation established)					
Lawns, open spaces, 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
Poor condition; grass cover on 50% or less of the area		68	79	86	89
Paved parking lots, roofs, driveways, etc. (excl. right-of- way)		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers (excl. right-of-way)		98	98	98	98
Gravel (incl. right-of-way)		76	85	89	91
Dirt (incl. right-of-way)		72	82	87	89
Paved with open ditches (incl. right-of-way)		83	89	92	93
	Average % impervious[b]				
Commercial and business areas	85	89	92	94	95
Industrial districts	72	81	88	91	93
Row houses, town houses, and residential with lots sizes 0.05 ha or less (0.12 acres or less)	65	77	85	90	92
Residential: average lot size					
0.1 ha (0.25 acres)	38	61	75	83	87
0.135 ha (0.33 acres)	30	57	72	81	86
0.2 ha (0.5 acres)	25	54	70	80	85
0.4 ha (1.0 acres)	20	51	68	79	84
0.8 ha (2.0 acres)	12	46	65	77	82
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 25- to 50-mm (1- to 2-in) sand or gravel mulch and basin borders)		96	96	96	96
Developing urban areas[c] (no vegetation established) Newly graded area		77	86	91	94

Table 5.4. Runoff Curve Numbers (Cont'd)

Cover Type		Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group A	B	C	D
Cultivated Agricultural Land: Fallow						
Straight row or bare soil			77	86	91	94
Conservation tillage		Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and tillage	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
		Good	62	71	78	81
	Contoured and terraces and conservation tillage	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and tillage	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
		Good	59	70	78	81
	Contoured and terraces and conservation tillage	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadows[e]	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
		Good	57	67	76	80
Noncultivated agricultural land						
Pasture or range	No Mechanical treatment[i]	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 - continuous grass, protected from grazing and generally mowed for hay			30	58	71	78

Table 5.4. Runoff Curve Numbers (Cont'd)

Cover Type	Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Forestland - grass or orchards - evergreen or Deciduous	Poor	55	73	82	86
	Fair	44	65	76	82
	Good	32	58	72	79
Brush - brush-weed-grass mixture with brush the major element[g]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[f]	48	65	73
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[f]	55	70	77
Woods - grass combination (orchard or tree farm)[h]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Farmsteads		59	74	82	86
Forest-range					
Herbaceous - mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen - mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon - juniper - pinyon, juniper, or both grass understory)	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sage-grass	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub - major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

a For land uses with impervious areas, curve numbers are computed assuming that 100 percent of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a CN of 98.

b Includes paved streets.

c Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area CN, the composite CN can be computed for any degree of development.

d For conservation tillage poor hydrologic condition, 5 to 20 percent of the surface is covered with residue (less than 850 kg/ha (760 lbs/acre) row crops or 350 kg/ha (310 lbs/acre) small grain). For conservation tillage good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 850 kg/ha (760 lbs/acre) row crops or 350 kg/ha (310 lbs/acre) small grain).

e Close-drilled or broadcast.
For noncultivated agricultural land:
Poor hydrologic condition has less than 25 percent ground cover density.
Fair hydrologic condition has between 25 and 50 percent ground cover density.
Good hydrologic condition has more than 50 percent ground cover density.
For forest-range.
Poor hydrologic condition has less than 30 percent ground cover density.
Fair hydrologic condition has between 30 and 70 percent ground cover density.
Good hydrologic condition has more than 70 percent ground cover density.

f Actual curve number is less than 30: use CN = 30 for runoff computations.

g CNs shown were computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

h Poor: < 50 percent ground cover.
Fair: 50 to 75 percent ground cover.
Good: > 75 percent ground cover.

i Poor: < 50 percent ground cover or heavily grazed with no mulch.
Fair: 50 to 75 percent ground cover and not heavily grazed.
Good: > 75 percent ground cover and lightly or only occasionally grazed.

5.2.5 Estimation of CN Values for Urban Land Uses

The CN table (Table 5.4) includes CN values for a number of urban land uses. For each of these, the CN is based on a specific percentage of imperviousness. For example, the CN values for commercial land use are based on an imperviousness of 85 percent. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus CN values of 39, 61, 74, and 80 are used for hydrologic soil groups A, B, C, and D, respectively. These are the same CN values for pasture in good condition. Thus the following equation can be used to compute a weighted CN:

$$CN_w = CN_p \left(1 - f\right) + f\left(98\right) \tag{5.23}$$

in which f is the fraction (not percentage) of imperviousness. To show the use of Equation 5.23, the CN values for commercial land use with 85 percent imperviousness are:

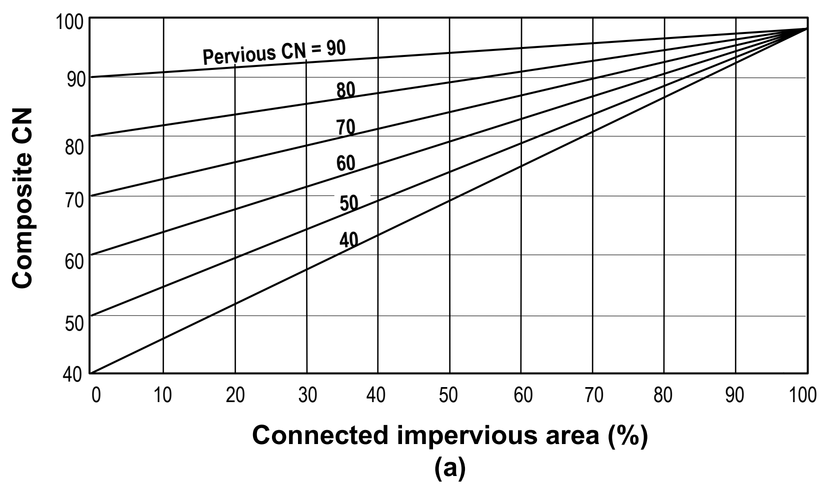
A soil: 39(0.15) + 98(0.85) = 89
B soil: 61(0.15) + 98(0.85) = 92
C soil: 74(0.15) + 98(0.85) = 94
D soil: 80(0.15) + 98(0.85) = 95

These are the same values shown in Table 5.4.

Equation 5.23 can be placed in graphical form (see Figure 5.2a). By entering with the percentage of imperviousness on the vertical axis at the center of the figure and moving horizontally to the pervious area CN, the composite CN can be read. The examples above for commercial land use can be used to illustrate the use of Figure 5.2a for 85 percent imperviousness. For a commercial land area with 60 percent imperviousness of a B soil, the composite CN would be:

$$CN_w = 61(0.4) + 98(0.6) = 83$$

The same value can be obtained from Figure 5.3a.



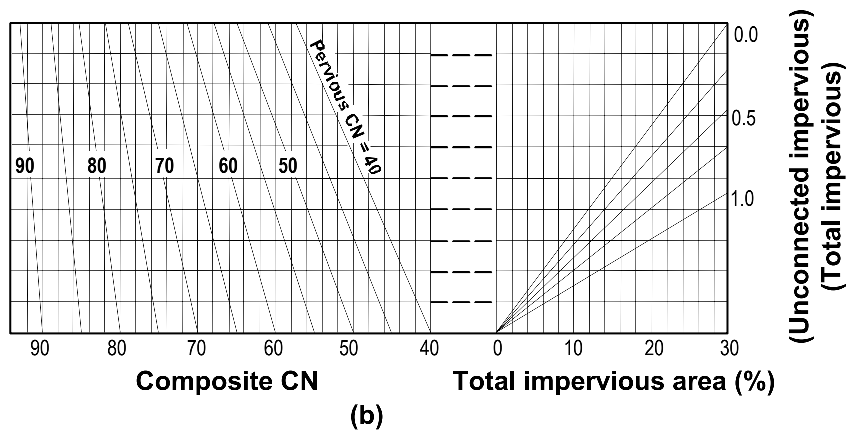


Figure 5.3. Composite curve number estimation:
(a) all imperviousness area connected to storm drains
(b) some imperviousness area not connected to storm drain

5.2.6 Effect of Unconnected Impervious Area on Curve Numbers

Many local drainage policies are requiring runoff that occurs from certain types of impervious land cover (i.e., rooftops, driveways, patios) to be directed to pervious surfaces rather than being connected to storm drain systems. Such a policy is based on the belief that disconnecting these impervious areas will require smaller and less costly drainage systems and lead both to increased ground water recharge and to improvements in water quality. If disconnecting some impervious surfaces will reduce both the peak runoff rates and volumes of direct flood runoff, credit should be given in the design of drainage systems. The effect of disconnecting impervious surfaces on runoff rates and volumes can be accounted for by modifying the CN.

There are three variables involved in the adjustment: the pervious area CN, the percentage of impervious area, and the percentage of the imperviousness that is unconnected. Because Figure 5.3a for computing composite CN values is based on the pervious area CN and the percentage of imperviousness, a correction factor was developed to compute the composite CN. The correction is a function of the percentage of unconnected imperviousness, which is shown in Figure 5.3b. The use of the correction is limited to drainage areas having percentages of imperviousness that are less than 30 percent.

As an alternative to Figure 5.3b, the composite curve number (CN_c) can be computed by:

$$CN_c = CN_p + \left(P_i/100\right)\left(98 - CN_p\right)\left(1 - 0.5R\right) \quad \text{for } P_i \le 30\% \tag{5.24}$$

where,

P_i = percent imperviousness
R = ratio of unconnected impervious area to the total impervious area.

Equation 5.24, like Figure 5.3b, is limited to cases where the total imperviousness (P_i) is less than 30 percent.

5.2.7 I_a/P Parameter

I_a/P is a parameter that is necessary to estimate peak discharge rates. I_a denotes the initial abstraction, and P is the 24-hour rainfall depth for a selected return period. For a given 24-hour rainfall distribution, I_a/P represents the fraction of rainfall that must occur before runoff begins.

5.2.8 Peak Discharge Estimation

The following equation can be used to compute a peak discharge with the SCS method:

$$q_p = q_u A Q \tag{5.25}$$

where,

q_p = peak discharge, m^3/s (ft^3/s)
q_u = unit peak discharge, $m^3/s/km^2/mm$ ($ft^3/s/\ mi^2/in$)
A = drainage area, km^2 (mi^2)
Q = depth of runoff, mm (in).

The unit peak discharge is obtained from the following equation, which requires the time of concentration (t_c) in hours and the initial abstraction/rainfall (I_a/P) ratio as input:

$$q_u = \alpha\, 10^{C_0 + C_1 \log t_c + C_2 [\log (t_c)]^2} \tag{5.26}$$

where,

C_o, C_1, and C_2 = regression coefficients given in Table 5.5 for various I_a/P ratios

α = unit conversion constant equal to 0.000431 in SI units and 1.0 in CU units.

The runoff depth (Q) is obtained from Equation 5.21 and is a function of the depth of rainfall P and the runoff CN. The I_a/P ratio is obtained directly from Equation 5.20.

Table 5.5. Coefficients for SCS Peak Discharge Method

Rainfall Type	I_a/P	C_0	C_1	C_2
I	0.10	2.30550	-0.51429	-0.11750
	0.20	2.23537	-0.50387	-0.08929
	0.25	2.18219	-0.48488	-0.06589
	0.30	2.10624	-0.45695	-0.02835
	0.35	2.00303	-0.40769	0.01983
	0.40	1.87733	-0.32274	0.05754
	0.45	1.76312	-0.15644	0.00453
	0.50	1.67889	-0.06930	0.0
IA	0.10	2.03250	-0.31583	-0.13748
	0.20	1.91978	-0.28215	-0.07020
	0.25	1.83842	-0.25543	-0.02597
	0.30	1.72657	-0.19826	0.02633
	0.50	1.63417	-0.09100	0.0
II	0.10	2.55323	-0.61512	-0.16403
	0.30	2.46532	-0.62257	-0.11657
	0.35	2.41896	-0.61594	-0.08820
	0.40	2.36409	-0.59857	-0.05621
	0.45	2.29238	-0.57005	-0.02281
	0.50	2.20282	-0.51599	-0.01259
III	0.10	2.47317	-0.51848	-0.17083
	0.30	2.39628	-0.51202	-0.13245
	0.35	2.35477	-0.49735	-0.11985
	0.40	2.30726	-0.46541	-0.11094
	0.45	2.24876	-0.41314	-0.11508
	0.50	2.17772	-0.36803	-0.09525

The peak discharge obtained from Equation 5.26 assumes that the topography is such that surface flow into ditches, drains, and streams is relatively unimpeded. Where ponding or wetland areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak discharge rate should be reduced to reflect this condition of increased storage. Values of the pond and swamp adjustment factor (F_p) are provided in Table 5.6. The adjustment factor values in Table 5.6 are a function of the percent of the total watershed area in ponds and wetlands. If the watershed includes significant portions of pond and wetland storage, the peak discharge of Equation 5.25 can be adjusted using the following:

$$q_a = q_p F_p \tag{5.27}$$

where,

q_a = adjusted peak discharge, m^3/s (ft^3/s).

Table 5.6. Adjustment Factor (F_p) for Pond and Wetland Areas

Area of Pond and Wetland (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

The SCS method has a number of limitations. When these conditions are not met, the accuracy of estimated peak discharges decreases. The method should be used on watersheds that are homogeneous in CN; where parts of the watershed have CNs that differ by 5, the watershed should be subdivided and analyzed using a hydrograph method, such as TR-20 (SCS, 1984). The SCS method should be used only when the CN is 50 or greater and the t_c is greater than 0.1 hour and less than 10 hours. Also, the computed value of I_a/P should be between 0.1 and 0.5. The method should be used only when the watershed has one main channel or when there are two main channels that have nearly equal times of concentration; otherwise, a hydrograph method should be used. Other methods should also be used when channel or reservoir routing is required, or where watershed storage is either greater than 5 percent or located on the flow path used to compute the t_c.

Example 5.4. A small watershed (17.6 ha) is being developed and will include the following land uses: 10.6 ha of residential (0.1 ha lots), 5.2 ha of residential (0.2 ha lots), 1.2 ha of commercial property (85 percent impervious), and 0.4 ha of woodland. The development will necessitate upgrading of the drainage of a local roadway at the outlet of the watershed. The peak discharge for a 10-year return period is determined using the SCS graphical method.

The weighted CN is computed using the CN values of Table 5.4:

Land Cover	Lot Size (ha)	Lot Size (acres)	Soil Group	5.2.8.1.1	Area (ha)	Area (acres)	A*CN (ha)	A*CN (acres)
Residential	0.2	0.5	B	70	5.2	12.8	364	896
Residential	0.1	0.25	B	75	4.6	11.4	345	855
Residential	0.1	0.25	C	83	6.0	14.8	498	1228
Commercial (85% Imp.)			C	94	1.2	3.0	113	282
Woodland (Good condition)			C	70	0.6	1.5	42	105
Total					17.6	43.5	1,362	3366

The weighted CN is:

Variable	Value in SI	Value in CU
$CN_w = \frac{\sum A * CN}{\sum A}$	$= \frac{1,362}{17.6} = 77.4\ (use\ 77)$	$= \frac{3,366}{43.5} = 77.4\ (use\ 77)$

The time of concentration is computed using the velocity method for conditions along the principal flowpath:

Conveyance Type	Slope (%)	K	Length (m)	V (m/s)	Length (ft)	V (ft/s)	T_t (h)
Woodland (overland)	2.3	0.152	25	0.23	82	0.76	0.03
Grassed waterway	2.1	0.457	275	0.66	902	2.19	0.12
Grassed waterway	1.8	0.457	250	0.61	820	2.02	0.11
Concrete-lined channel	1.8	-	50	4.62	164	15.1	0.00
			600		1968		0.26

The velocity was computed for the concrete-lined channel using Manning's equation, with n = 0.013 and hydraulic radius of 0.3 m (1ft). The sum of the travel times for the principal flowpath is 0.26 hours.

The rainfall depth is obtained from an IDF curve for the locality using a storm duration of 24 hours and a 10-year return period. (Note that the t_c is not used to find the rainfall depth when using the SCS graphical method. A storm duration of 24 hours is used.) For this example, a 10-year rainfall depth of 122 mm (4.8 in) is assumed. For a CN of 77, S equals 76 mm (3.0 in) and I_a equals 15 mm (0.6 in). Thus, I_a/P is 0.12. The rainfall depth is computed with Equation 5.21:

Variable	Value in SI	Value in CU
$Q = \frac{(P - 0.2S)^2}{P + 0.8\ S}$	$= \frac{(122\ - 0.2(76))^2}{122\ + 0.8(76)} = 62\ mm$	$= \frac{(4.8\ - 0.2(3.0))^2}{4.8\ + 0.8(3.0)} = 2.45\ in$

The unit peak discharge is computed with Equation 5.26 by interpolating c_0 , c_1, and c_2 from Table 5.5 using a type II distribution. The peak discharge is also calculated as follows.

Variable	SI Unit	**CU Unit**
$q_u = 10^{2.5444-0.61587\ \log(0.26) - 0.15928[\log(0.26)]}$	$= (0.000431)\ 10^{2.85}$ $= 0.305\ m^3/s/km^2/mm$	$= (1)\ 10^{2.85}$ $= 708\ ft^3/s/mi^2/in$
$q_p = q_u A Q$	$= 0.305\ (0.176\ km^2)(62\ mm)$ $= 3.3\ m^3/s$	$= 708\ (0.068\ mi^2)\ (2.46\ in)$ $= 120\ ft^3/s$

5.3 RATIONAL METHOD

One of the most commonly used equations for the calculation of peak discharges from small areas is the rational formula. The rational formula is given as:

$$Q = \frac{1}{\alpha}\ C\ i\ A \qquad (5.28)$$

where,

- Q = the peak flow, m^3/s (ft^3/s)
- i = the rainfall intensity for the design storm, mm/h (in/h)
- A = the drainage area, ha (acres)
- C = dimensionless runoff coefficient assumed to be a function of the cover of the watershed and often the frequency of the flood being estimated
- α = unit conversion constant equal to 360 in SI units and 1 in CU units.

5.3.1 Assumptions

The assumptions in the rational formula are as follows:

1. The drainage area should be smaller than 80 hectares (200 acres).

2. The peak discharge occurs when the entire watershed is contributing.

3. A storm that has a duration equal to t_c produces the highest peak discharge for this frequency.

4. The rainfall intensity is uniform over a storm time duration equal to the time of concentration, t_c. The time of concentration is the time required for water to travel from the hydrologically most remote point of the basin to the outlet or point of interest.

5. The frequency of the computed 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 peak discharge.

5.3.2 Estimating Input Requirements

The runoff coefficient, C, is a function of ground cover. Some tables of C provide for variation due to slope, soil, and the return period of the design discharge. Actually, C is a volumetric coefficient that relates the peak discharge to the "theoretical peak" or 100 percent runoff, occurring when runoff matches the net rain rate. 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 5.7. Should the basin contain varying amounts of different covers, a weighted runoff coefficient for the entire basin can be determined as:

$$\text{Weighted } C = \frac{\sum C_i A_i}{A} \tag{5.29}$$

where,

C_i = runoff coefficient for cover type i that covers area A_i

A = total area.

5.3.3 Check for Critical Design Condition

When the rational method is used to design multiple drainage elements (i.e. inlets and pipes), the design process proceeds from upstream to downstream. For each design element, a time of concentration is computed, the corresponding intensity determined, and the peak flow computed. For pipes that drain multiple flow paths, the longest time of concentration from all of the contributing areas must be determined. If upstream pipes exist, the travel times in these pipes must also be included in the calculation of time of concentration.

In most cases, especially as computations proceed downstream, the contributing area with the longer time of concentration also contributes the greatest flow. Taking the case of two contributing areas, as shown in Figure 5.3a, the longest time of concentration of the two areas is used to determine the time of concentration for the combined area. When the rainfall intensity corresponding to this time of concentration is applied to the rational equation, as shown below, for the combined area and runoff coefficient, the appropriate design discharge, Q, results.

$$Q = \frac{1}{\alpha}(C_1 A_1 + C_2 A_2)\, i_1 \tag{5.30}$$

However, it may be possible for the larger contributing flows to be generated from the contributing area with a shorter time of concentration. If this occurs, it is also possible that, if the longer time of concentration is applied to the combined drainage area, the resulting design flow would be an underestimate. Therefore, a check for a critical design condition must be made.

$$Q' = \frac{1}{\alpha}\left(C_1 A_1 \frac{t_2}{t_1} + C_2 A_2\right) i_2 \tag{5.31}$$

where,

Q' = design check discharge

t_1 = time of concentration for area 1

t_2 = time of concentration for area 2.

Table 5.7. Runoff Coefficients for Rational Formula (ASCE, 1960)

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown area	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%	0.05-0.10
Sandy soil, average, 2 to 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 to 7%	0.18-0.22
Heavy soil, steep, > 7%	0.25-0.35
Streets:	
Asphalt	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

If Q’ > Q, Q’ should be used for design; otherwise Q should be used. Equation 5.31 uses the rainfall intensity for the contributing area with the shorter time of concentration (area 2) and reduces the contribution of area 1 by the ratio of the times of concentration. This ratio approximates the fraction of the area that would contribute within the shorter duration. This is equivalent to reducing the contributing area as shown by the dashed line in Figure 5.4.

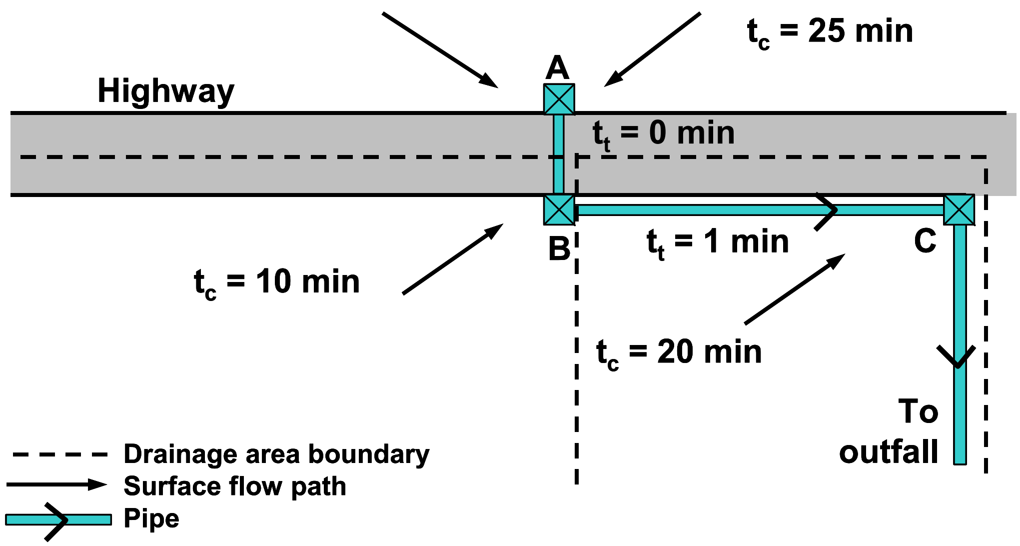


Figure 5.4. Storm drain system schematic

Example 5.5. 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 road safety during rainstorms. The drainage area above the crossing is 43.7 ha (108 acres). The return period of the design storm is to be 25 years as determined by local authorities. The engineer must determine the maximum discharge that the culvert must pass for the indicated design storm.

The current land use consists of 21.8 ha (53.9 acres) of parkland, 1.5 ha (3.7 acres) of commercial property that is 100 percent impervious, and 20.4 ha (50.4 acres) of single-family residential housing. The principal flow path includes 90 m (295 ft) of short grass at 2 percent slope, 300 m (985 ft) of grassed waterway at 2 percent slope, and 650 m (2,130 ft) of grassed waterway at 1 percent slope. The following steps are used to compute the peak discharge with the rational method:

1. **Compute a Weighted Runoff Coefficient**: The tabular summary below uses runoff coefficients from Table 5.7. The average value is used for the parkland and the residential areas, but the highest value is used for the commercial property because it is completely impervious.

Description	C Value	SI Unit		CU Unit	
		Area (ha)	C_iA_i	Area (acres)	C_iA_i
Park	0.20	21.8	4.36	53.9	10.8
Commercial (100% impervious)	0.95	1.5	1.43	3.7	3.5
Single-family	0.40	20.4	8.16	50.4	20.2
Total		43.7	13.95	108.0	34.5

Equation 5.29 is used to compute the weighted C:

Variable	Value in SI	Value in CU
$\text{Weighted } C = \frac{\sum C_i A_i}{A}$	$= \frac{13.95}{43.7} = 0.32$	$= \frac{34.5}{108.0} = 0.32$

2. **Intensity**: The 25-year intensity is taken from an intensity-duration-frequency curve for Memphis. To obtain the intensity, the time of concentration, t_c, must first be estimated. In this example, the velocity method for t_c is used to compute t_c:

Flow Path	Slope(%)	SI Unit		CU Unit	
		Length (m)	Velocity (m/s)	Length (ft)	Velocity (ft/s)
Overland (Short grass)	2	90	0.30	295	1.0
Grassed waterway	2	300	0.64	985	2.1
Grassed waterway	1	650	0.46	2,130	1.5

The time of concentration is estimated as:

Variable	Value in SI	Value in CU
$T_c = \sum\left(\frac{L}{V}\right)$	$= \frac{90\ m}{0.3\ m/s} + \frac{300\ m}{0.64\ m/s} + \frac{650\ m}{0.46\ m/s}$ $= 2{,}180\ s = 36\ min$	$= \frac{295\ ft}{1.0\ ft/s} + \frac{985\ ft}{2.1\ ft/s} + \frac{2{,}130\ ft}{1.5\ ft/s}$ $= 2{,}180\ s = 36\ min$

The intensity is obtained from the IDF curve for the locality using a storm duration equal to the time of concentration:

$$i = 85 \text{ mm/h } (3.35 \text{in/h})$$

3. **Area (A)**: Total area of drainage basin, A = 43.7 ha (108 acres)

4. **Peak Discharge (Q)**:

Variable	Value in SI	Value in CU
$Q = \frac{1}{\alpha} CiA$	$= \frac{(0.32)(85)(43.7)}{360} = 3.3\ m^3/s$	$= \frac{(0.32)(3.35)(108)}{1} = 116 ft^3/s$

5.4 INDEX FLOOD METHOD

Other methods exist for determining peak flows for various exceedence frequencies using regional methods where no data are available. The USGS index-flood method is representative of this group.

5.4.1 Procedure for Analysis

The index-flood method of regional analysis described by Dalrymple (1960) was used extensively in the 1960s and early 1970s. 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 the mean annual flood) and the second involves developing the relation between the drainage basin characteristics (usually the drainage area) and the mean annual flood.

The following steps are used to develop a regional flood frequency curve by the index-flood method:

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 positions, and compute and plot frequency curves using the best standard distribution fit for each gage.

5. Determine the mean annual flood 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. In some cases, the 2- or 10-year flood is used as the index flood.

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.

   e. Test for homogeneity by also plotting on this graph, envelope curves determined from Table 5.8, 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 5.8 applies only to homogeneity tests of the 10-year floods.) Return periods that fail this homogeneity test should be eliminated from the regional analysis.

7. Using actual flood data, compute the ratio of each flood to the index flood, $Q_{2.33}$, 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, $T_r = (n+1)/m$. (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.

Table 5.8. Upper and Lower Limit Coordinates of Envelope Curve for Homogeneity Test (Dalrymple, 1960)

	Return Period Limits, T_r (yrs)	
Effective Length of Record, L_E (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

Example problems illustrating the index-flood method are contained in Dalrymple (1960), Sanders (1980), and numerous hydrology textbooks.

5.4.2 Other Considerations

As pointed out by Benson (1962), the index-flood method has some limitations that 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 Chapter 2, the larger the drainage area, the flatter the frequency curve will be. This effect is most noticeable at the higher return periods.

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.

5.5 PEAK DISCHARGE ENVELOPE CURVES

Design storms are hypothetical constructs and have never occurred. Many design engineers like to have some assurance that a design peak discharge is unlikely to occur over the design life of a project. This creates an interest in comparing the design peak to actual peaks of record.

Crippen and Bue (1977) developed envelope curves for the conterminous United States, with 17 regions delineated as shown in Figure 5.5. Maximum floodflow data from 883 sites that have drainage areas less than 25,900 km^2 (10,000 mi^2) were plotted versus drainage area and upper envelope curves constructed. The curves for the 17 regions were fit to the following logarithmic polynomial model:

$$q_{envlpe} = K_1 A^{K_2} \left[L + A^{0.5}\right]^{K_3} \quad (5.32)$$

where,

q_{envlpe} = maximum flood flow envelope, m^3/s (ft^3/s)
L = length constant, 8.0 km (5.0 mi)
A = drainage area, km^2 (mi^2).

Table 5.9 gives the values of the coefficients (K_1, K_2, and K_3 of Equation 5.32) and the upper limit on the drainage area for each region. The curves are valid for drainage areas greater than 0.25 km^2 (0.1 mi^2). Crippen and Bue did not assign an exceedence probability to the flood flows used to fit the curves, so a probability cannot be given to values estimated from the curves.

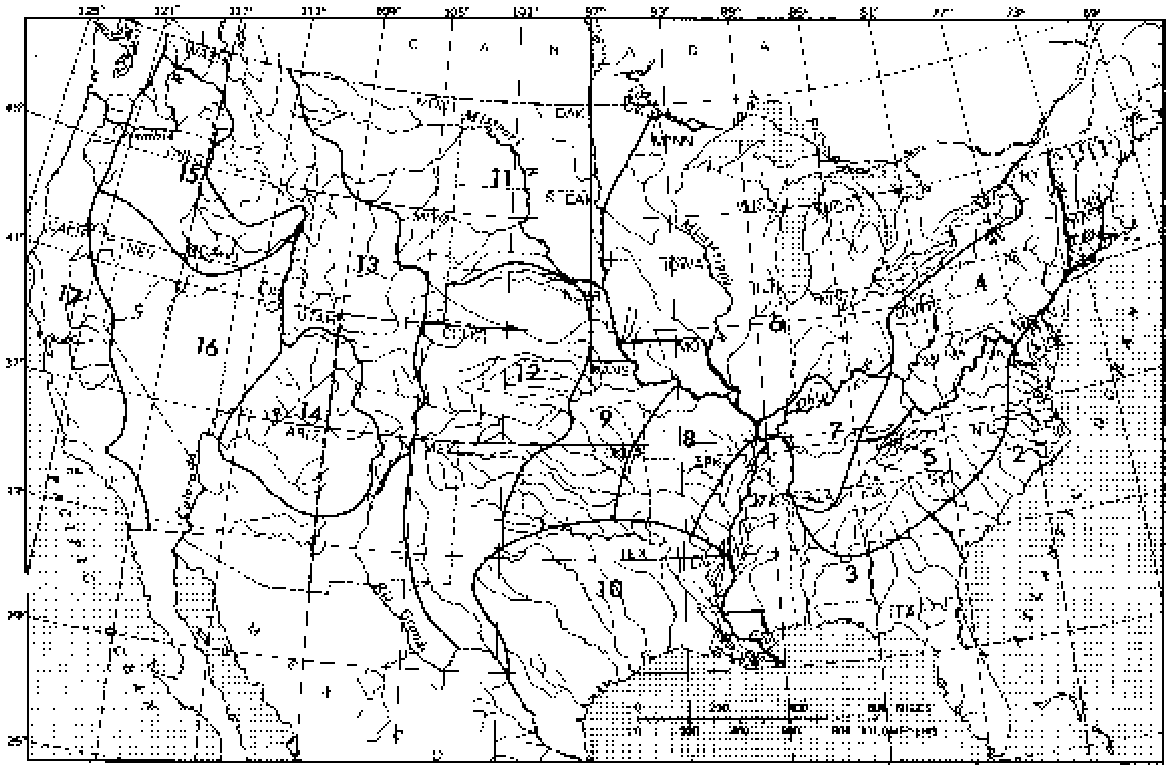


Figure 5.5. Map of the conterminous United States showing flood-region boundaries

Table 5.9. Coefficients for Peak Discharge Envelope Curves

(a) SI Unit

Region	Upper limit (km²)	Coefficients		
		K_1	K_2	K_3
1	26,000	469	0.895	-1.082
2	7,800	584	0.770	-0.897
3	26,000	1229	0.924	-1.373
4	26,000	929	0.938	-1.327
5	26,000	2939	0.838	-1.354
6	26,000	1517	0.937	-1.297
7	26,000	1142	0.883	-1.352
8	26,000	954	0.954	-1.357
9	26,000	1815	0.849	-1.368
10	2,600	1175	1.116	-1.371
11	26,000	917	0.919	-1.352
12	18,100	1944	0.935	-1.304
13	26,000	1504	0.873	-1.338
14	26,000	215	0.710	-0.844
15	50	2533	1.059	-1.572
16	2,600	1991	1.029	-1.341
17	26,000	1724	1.024	-1.461

(b) CU Unit

Region	Upper limit (mi²)	Coefficients		
		K_1	K_2	K_3
1	10,000	23200	0.895	-1.082
2	3,000	28000	0.770	-0.897
3	10,000	54400	0.924	-1.373
4	10,000	42600	0.938	-1.327
5	10,000	121000	0.838	-1.354
6	10,000	70500	0.937	-1.297
7	10,000	49100	0.883	-1.352
8	10,000	43800	0.954	-1.357
9	10,000	75000	0.849	-1.368
10	1,000	62500	1.116	-1.371
11	10,000	40800	0.919	-1.352
12	7,000	89900	0.935	-1.304
13	10,000	64500	0.873	-1.338
14	10,000	10000	0.710	-0.844
15	19	116000	1.059	-1.572
16	1,000	98900	1.029	-1.341
17	10,000	80500	1.024	-1.461

CHAPTER 6

DESIGN HYDROGRAPHS

In discussing the concept of hydrographs, it is helpful to discuss the issue in terms of a fundamental concept of systems theory. A system can be viewed as consisting of three functions: the input function, the transfer function, and the output function. The rainfall hyetograph is the input function and the total runoff hydrograph is the output function. In this chapter, the transfer function will be represented by a unit hydrograph.

A purpose of hydrograph analysis is to analyze measured rainfall and runoff data to obtain an estimate of the transfer function. Once the transfer function has been developed, it can be used with both design storms and measured rainfall hyetographs to compute (synthesize) the expected runoff. The resulting runoff hydrograph can then be used for design purposes. Unit hydrographs (UH) can be developed for a specific watershed or for general use on watersheds where data are not available to develop a unit hydrograph specifically for that watershed; those of the latter type are sometimes referred to as synthetic unit hydrographs.

While a number of conceptual frameworks are available for hydrograph analysis, the one presented herein will involve the following: (1) the separation of the rainfall hyetograph into three parts, (2) the separation of the runoff hydrograph into two parts, and (3) the identification of the unit hydrograph as the transfer function.

The rainfall hyetograph is separated into three time-dependent functions: the initial abstraction, the loss function, and the rainfall excess. These functions are shown in Figure 6.1 using a standard convention of inverting the hyetograph. The initial abstraction is that part of the rainfall that occurs prior to the start of direct runoff (which is defined below). The rainfall excess is that part of the rainfall that appears as direct runoff. The loss function is that part of the rainfall that occurs after the start of direct runoff, but does not appear as direct runoff. The process is sometimes conceptualized as a two-part separation of the rainfall, with the initial abstraction being included as part of the loss function. The three components are used here for clarity and to emphasize the differences between the important processes of the hydrologic cycle.

The runoff hydrograph is conceptually separated into two parts: direct runoff and base flow, as shown in Figure 6.1. The direct runoff is the storm runoff that results from rainfall excess; the volumes of rainfall excess and direct runoff must be equal. The transfer function, or unit hydrograph, is the function that transforms the rainfall excess into the direct runoff. For the purpose of our conceptual framework, base flow is the runoff that has resulted from an accumulation of water in the watershed from past storm events and would appear as stream flow even if the rain for the current storm event had not occurred. It also includes increases to ground-water discharge that occurs during and after storm events.

Having completed the analysis phase through the development of a unit hydrograph, the results of the analysis can be used to synthesize hydrographs at ungauged locations (i.e., at locations where data to conduct analyses are not available). In the synthesis phase, a rainfall excess hyetograph and a unit hydrograph are used to compute a direct runoff hydrograph. The process of transforming the rainfall excess into direct runoff using the unit hydrograph is called convolution. The rainfall hyetograph can be either a synthetic design storm or a measured storm event.

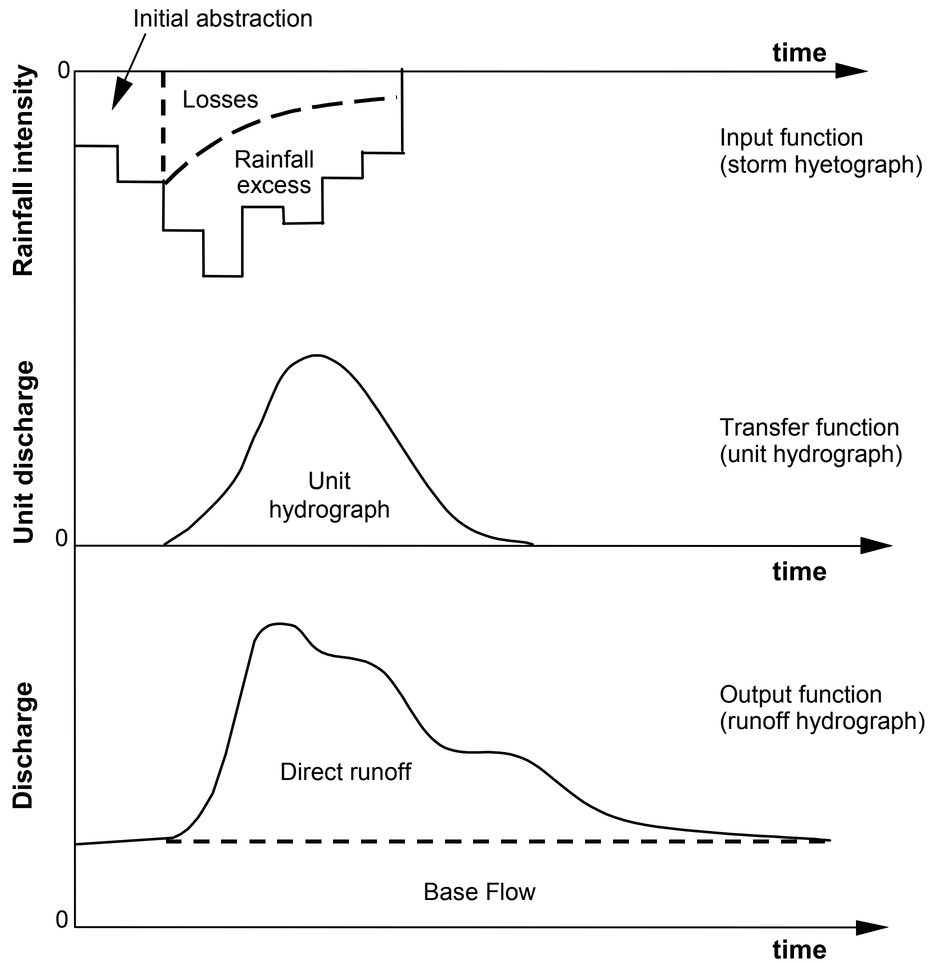


Figure 6.1. Rainfall/runoff as the system process

In summary, in the analysis phase, the hyetograph and hydrograph are known and the unit hydrograph is estimated. In the synthesis phase, a hyetograph is used with a unit hydrograph to compute a runoff hydrograph.

In performing a hydrograph analysis for a basin with gauged rainfall and runoff data, it is common to begin by separating the base flow from the total runoff hydrograph. This is usually the first step because base flow is usually a smooth function and it can probably be estimated more accurately than the loss function. The direct runoff hydrograph equals the difference between the total hydrograph and the base flow.

Having computed the base flow and direct runoff hydrographs, the volume of direct runoff can be computed as the volume under the direct runoff hydrograph. Then the initial abstraction is delineated, if the initial abstraction is to be handled separately from the other losses. Finally, the losses are separated from the total rainfall hyetograph such that the volume of rainfall excess equals the volume of direct runoff.

6.1 UNIT HYDROGRAPH ANALYSIS

In Chapter 2, 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. At this time, it is not possible to accurately model this relationship mathematically and to predict the response of a watershed to any precipitation event. 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; therefore, a more practical tool is necessary. Highway designers can use unit hydrograph techniques to approximate the rainfall-runoff response of typical watersheds. These methods do not require as much data and are usually sufficiently accurate for highway stream-crossing design.

6.1.1 Assumptions

A stage hydrograph is a plot or tabulation of the water level versus time. A runoff hydrograph is a plot of discharge rate 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, the corresponding runoff hydrograph would be the response of the given watershed to a 1-hour storm. Figure 6.2 illustrates the direct runoff hydrograph from a rainfall of 1-hour duration. The duration of the runoff, which is called the time base of the hydrograph, is 4.25 hours, which is much greater than the duration of rainfall excess.

Suppose that the same watershed was subjected to another storm that was the same in all respects except that the rainfall excess was twice as intense. The unit hydrograph technique assumes that the time base of the runoff hydrograph remains unchanged for equal duration storms and that the ordinates are directly proportional to the amount of rainfall excess. In this particular case, the ordinates are twice as high as for the previous storm (see Figure 6.3). This illustrates the linearity assumption that underlies unit hydrograph theory. The amount of direct runoff is directly proportional to the amount of rainfall excess.

Now suppose that immediately after the 1-hour storm shown in Figure 6.2, another 1-hour storm of exactly the same intensity and spatial distribution occurred. Unit hydrograph theory assumes that the second storm by itself would produce an identical direct runoff hydrograph that is independent of antecedent conditions. It would be exactly the same as the first hydrograph and would be additive to the first except lagged by 1 hour. The resulting total direct runoff hydrograph would be as illustrated in Figure 6.4. The time base of the resulting hydrograph is 5.25 hours.

The above examples serve to illustrate the underlying assumptions applicable to unit hydrograph techniques. Johnston and Cross (1949) list the three basic assumptions that are fundamental to unit hydrograph theory:

1. For a given drainage basin, the duration of direct runoff is essentially constant for all uniform-intensity storms of the same duration, regardless of differences in the total volume of the direct runoff.

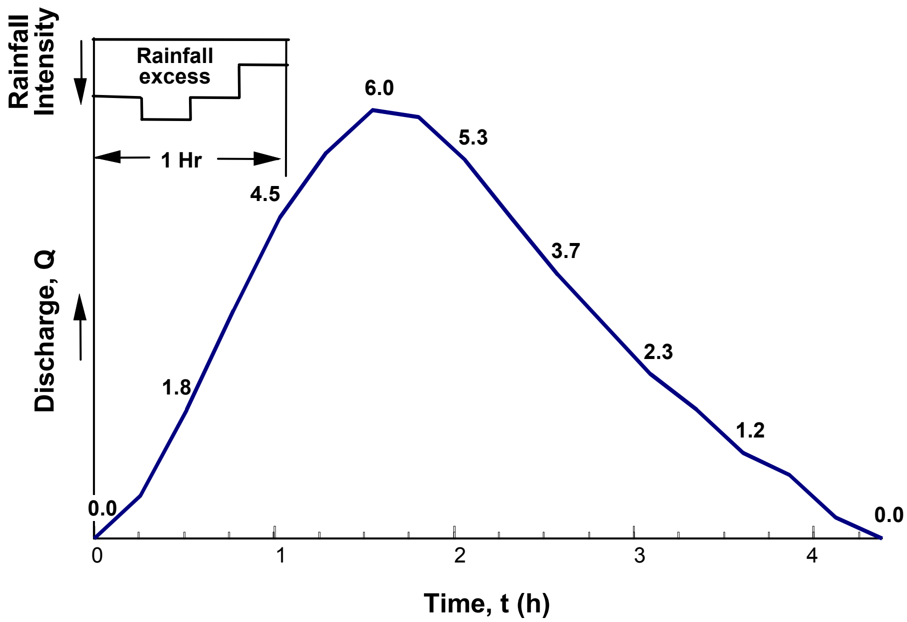


Figure 6.2. Runoff hydrograph for a 1-hour storm

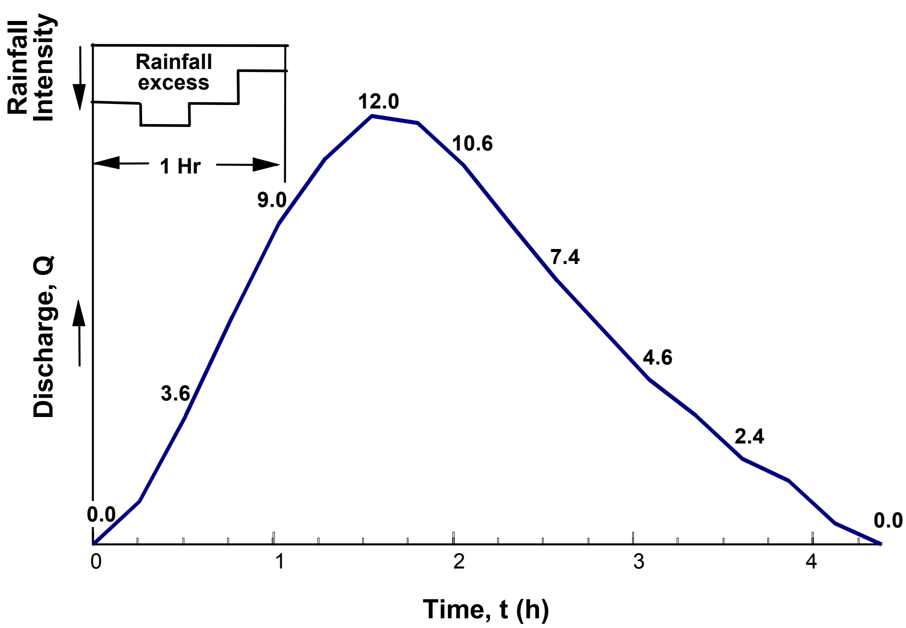


Figure 6.3. Runoff hydrograph for a 1-hour storm with twice the intensity as that in Figure 6.2

2. For a given drainage basin, two distributions of rainfall excess that have the same duration but different volumes will produce distributions of direct runoff that are of the same duration but with ordinates that are proportional to the volumes of rainfall excess.

3. The time distribution of direct runoff from a given storm duration is independent of concurrent runoff from antecedent storms.

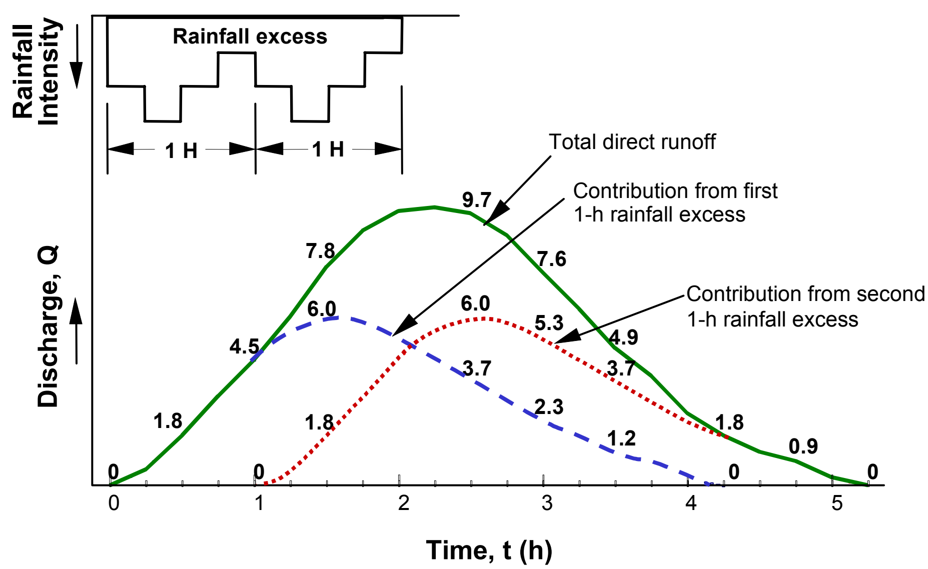


Figure 6.4. Runoff hydrograph for two successive 1-hour storms

6.1.2 Unit Hydrograph Definitions

A unit hydrograph (UH) is defined as the direct runoff hydrograph resulting from a rainfall event that has uniform temporal and spatial distributions and the volume of direct runoff represented by the area under the unit hydrograph is equal to one unit of direct runoff from the drainage area. In CU units, a unit depth is 1 inch; in SI units, a unit depth is 1 mm. Thus, when a unit hydrograph is shown with units of cubic meters per second (m^3/s), it is implied that the ordinates are $m^3/s/mm$ of direct runoff and when it is shown with units of cubic feet per second (ft^3/s), it is implied that the ordinates are $ft^3/s/in$.

A different unit hydrograph exists for each 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 affected by the temporal and spatial distributions of the actual rainfall excess. In other words, two rainfall events with different distributions over the drainage area may give different unit hydrographs even if their respective durations are identical. Variations of the temporal and spatial distributions of rainfall contribute to variations in computed unit hydrographs for different storm events on the same watershed.

Several types of unit hydrographs can be developed. A D-hour (or D-minute) unit hydrograph is the hydrograph that results from a storm with a constant rainfall excess of 1 mm (1 in) spread uniformly over a duration of D hours (or D minutes). An instantaneous unit hydrograph (IUH) is a special case of the D-hour UH with the duration of rainfall excess being infinitesimally small; for such a UH to have a volume of 1 mm (1 in), the intensity of the instantaneous UH is obviously not finite. The dimensionless unit hydrograph, which is a third form, is a direct runoff hydrograph whose ordinates are given as ratios of the peak discharge and whose time axis is defined as the ratio of the time to peak (i.e., a dimensionless UH with an axis system of q/q_p versus t/t_p, where q_p is the discharge rate at the time to peak t_p). Before a dimensionless UH can be used, it must be converted to a D-hour UH.

The key to analyzing unit hydrographs is to select the correct rainfall events. The chosen storms must be representative of the temporal and spatial distribution of rainfall that is characteristic of storms resulting in peak discharges of the magnitudes and frequency selected for design.

6.1.3 Convolution

The process by which the design storm is combined with the unit hydrograph to produce the direct runoff hydrograph is called convolution. Conceptually, it is a process of multiplication, translation with time, and addition. That is, the first burst of rainfall excess of duration D is multiplied by the ordinates of the unit hydrograph, the UH is then translated a time length of D, and the next D-hour burst of rainfall excess is multiplied by the UH. After the UH has been translated for all D-hour bursts of rainfall excess, the results of the multiplications are summed for each time interval. This process of multiplication, translation, and addition is the means of deriving a design runoff hydrograph from the rainfall excess and the UH.

The convolution process is best introduced using some simple examples that illustrate the multiplication-translation-addition operations. First, consider a burst of rainfall excess of 1 mm (1 in) that occurs over a period D. Assuming that the UH consists of two ordinates, 0.4 and 0.6, the direct runoff is computed by multiplying the rainfall excess burst by the UH; this is presented graphically as in Figure 6.5a. It is important to note that the volume of direct runoff equals the volume of rainfall excess, which in this case is 1 mm (1 in). Thus, the runoff hydrograph from the 1-mm (1-in) storm in Figure 6.5a is the D-hour unit hydrograph.

If 2 mm (2 in) of rainfall excess occurs over a period of D, the direct runoff volume must be 2 mm (2 in). Using the same UH as the previous example, the resulting runoff hydrograph is shown in Figure 6.5b. In both this example and the previous example, computation of the runoff hydrograph consisted solely of multiplication; the translation and addition parts of the convolution process were not necessary because the rainfall excess occurred over a single time interval of D.

To illustrate the multiplication-translation-addition operation, consider 2 mm (2 in) of rainfall excess that occurs uniformly over a period 2D (Figure 6.5c). In this case, the direct runoff will have a volume of 2 mm (2 in), but the time distribution of direct runoff will differ from that of the previous problem because the time distribution of rainfall excess is different. Figure 6.5c shows the multiplication-translation-addition operation. In this case, the time base of the runoff hydrograph is 3 time units (i.e., 3D). In general, the time base of the runoff (t_{bRO}) is given by:

$$t_{bRO} = t_{bPE} + t_{bUH} - 1 \tag{6.1}$$

in which t_{bPE} and t_{bUH} are the time bases of the rainfall excess and unit hydrograph, respectively. For the example above, both t_{bPE} and t_{bUH} equal 2D, and, therefore, according to Equation 6.1 t_{bRO} equals 3D time units.

One more example should illustrate the convolution process. In Figure 6.5d, the volume of rainfall excess equals 3 mm (3 in) with 2 mm (2 in) occurring in the first time unit. The computation of the runoff hydrograph is shown in Figure 6.5d. In this case, the second ordinate of the runoff hydrograph is the sum of 2 mm (2 in) times the second ordinate of the UH and 1 mm (1 in) times the first ordinate of the translated UH:

$$2(0.6) + 1(0.4) = 1.6$$

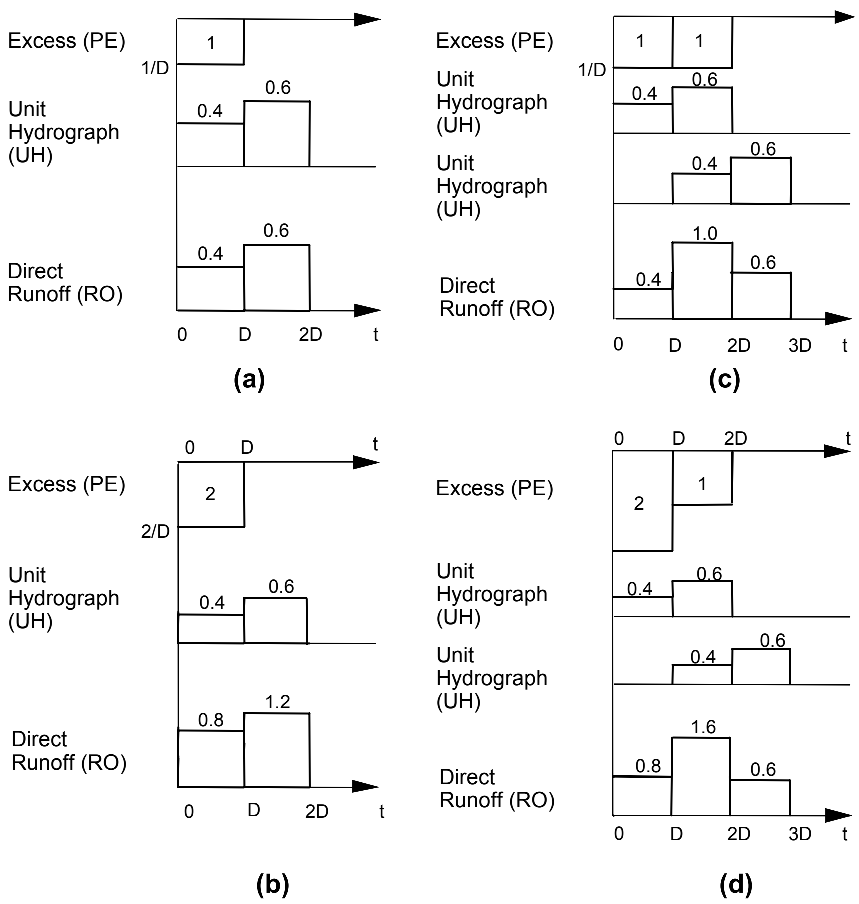


Figure 6.5. Convolution: a process of multiplication-translation-addition

For the case where t_{bUH} = 5 and t_{bPE} = 3, convolution can be presented by the following equations for computing the direct runoff Q(i) from the rainfall excess P(i) and the unit hydrograph U(i):

$$
\begin{aligned}
Q(1) &= P(1)U(1) \\
Q(2) &= P(1)U(2) + P(2)U(1) \\
Q(3) &= P(1)U(3) + P(2)U(2) + P(3)U(1) \\
Q(4) &= P(1)U(4) + P(2)U(3) + P(3)U(2) \\
Q(5) &= P(1)U(5) + P(2)U(4) + P(3)U(3) \\
Q(6) &= P(2)U(5) + P(3)U(4) \\
Q(7) &= P(3)U(5)
\end{aligned}
$$

The number of ordinates in the direct runoff distribution is computed with an equation similar to Equation 6.1:

$$t_{bro} = t_{bPE} + t_{bUH} - 1 = 3 + 5 - 1 = 7$$

Example 6.1(SI). Convolution can be illustrated using a 15-minute unit hydrograph for a 2.268 km^2 watershed. The duration of rainfall excess is 45 minutes, with intensities of 40, 80, and 60 mm/h for the three 15-minute increments. The unit hydrograph has a time base of 2 hours, with 15-minute ordinates of (0.0, 0.12, 0.55, 0.67, 0.63, 0.29, 0.18, 0.08) m^3/s/mm. To show that this is a unit hydrograph with an equivalent depth of 1 mm, the trapezoidal rule can be used to compute the depth:

$$
\begin{aligned}
Depth &= \Delta t\,(1mm)\sum_{i=1}^{n} U_i \\
&= (0.25\,h)(1mm)(0 + 0.12 + 0.55 + 0.67 + 0.63 + 0.29 + 0.18 + 0.08) \\
&\times \left(\frac{3600\,s}{h}\right)\left(\frac{1}{2.268\,km^2}\right)\left(\frac{1\,km}{1000\,m}\right)^2\left(\frac{1000\,mm}{1\,m}\right) \\
&= 1mm
\end{aligned}
$$

Before the rainfall excess is convolved with the unit hydrograph, the ordinates must be converted from intensities (mm/h) to depths (mm) by multiplying the intensities by the time increment of 0.25 hours. Thus, the ordinates of the rainfall excess expressed as depths are 10, 20, and 15 mm. Since there are three ordinates of rainfall excess and seven non-zero ordinates on the unit hydrograph, Equation 6.1 would indicate that the direct runoff hydrograph will have nine nonzero ordinates. The convolution is performed as follows:

$$
\begin{aligned}
Q_1 &= P_1U_1 &&= 10\,(0.12) &&= 1.20 \\
Q_2 &= P_1U_2 + P_2U_1 &&= 10\,(0.55) + 20\,(0.12) &&= 7.90 \\
Q_3 &= P_1U_3 + P_2U_2 + P_3U_1 &&= 10\,(0.67) + 20\,(0.55) + 15\,(0.12) &&= 19.50 \\
Q_4 &= P_1U_4 + P_2U_3 + P_3U_2 &&= 10\,(0.63) + 20\,(0.67) + 15\,(0.55) &&= 27.95 \\
Q_5 &= P_1U_5 + P_2U_4 + P_3U_3 &&= 10\,(0.29) + 20\,(0.63) + 15\,(0.67) &&= 25.55 \\
Q_6 &= P_1U_6 + P_2U_5 + P_3U_4 &&= 10\,(0.18) + 20\,(0.29) + 15\,(0.63) &&= 17.05 \\
Q_7 &= P_1U_7 + P_2U_6 + P_3U_5 &&= 10\,(0.08) + 20\,(0.18) + 15\,(0.29) &&= 8.75 \\
Q_8 &= P_2U_7 + P_3U_6 &&= 20\,(0.08) + 15\,(0.18) &&= 4.30 \\
Q_9 &= P_3U_7 &&= 15\,(0.08) &&= \underline{1.20} \\
& && && 113.40
\end{aligned}
$$

in which Q_i is the i[th] ordinate of the direct runoff hydrograph, P_i is the i[th] ordinate of the rainfall excess, and U_i is the i[th] ordinate of the unit hydrograph. The sum of the ordinates of the direct runoff hydrograph is 113.4 m^3/s. Thus, the depth of direct runoff is:

$$\Delta t \sum_{i=1}^{9} Q_i = (0.25\ h)\left(113.4\ \frac{m^3}{s}\right)\left(\frac{3600\ s}{h}\right)\left(\frac{1}{2.268\ km^2}\right)\left(\frac{1\ km^2}{10^6\ m^2}\right)\left(\frac{1000\ mm}{1\ m}\right)$$

$$= 45\ mm$$

Since the rainfall excess has a depth of 45 mm, then the direct runoff hydrograph must have a depth of 45 mm. The volume of direct runoff equals the depth of 45 mm times the drainage area.

Example 6.1(CU). Convolution can be illustrated using a 15-minute unit hydrograph for a 0.88 mi^2 watershed. The duration of rainfall excess is 45 minutes, with intensities of 1.6, 3.2, and 2.4 in/h for the three 15-minute increments. The unit hydrograph has a time base of 2 hours, with 15-minute ordinates of (0.0, 108, 493, 601, 565, 260, 161, 72) ft^3/s/in. To show that this is a unit hydrograph with an equivalent depth of 1 inch, the trapezoidal rule can be used to compute the depth:

$$Depth = \Delta t\ (1 in) \sum_{i=1}^{n} U_i$$

$$= 0.25\,h\ (1\ in)\ (0+108+493+601+565+260+161+72)\ \frac{ft^3}{s\ in}$$

$$\times \left[\frac{3600\ s}{h}\right]\left[\frac{1}{0.88\,mi^2}\right]\left[\frac{1\ mi}{5280\ ft}\right]^2\left[\frac{12\ in}{1\ ft}\right]$$

$$= 1\ in$$

Before the rainfall excess is convolved with the unit hydrograph, the ordinates must be converted from intensities (in/h) to depths (in) by multiplying the intensities by the time increment of 0.25 hour. Thus, the ordinates of the rainfall excess expressed as depths are 0.4, 0.8, and 0.6 in. Since there are three ordinates of rainfall excess and seven non-zero ordinates on the unit hydrograph, Equation 6.1 would indicate that the direct runoff hydrograph will have nine nonzero ordinates. The convolution is performed as follows:

$Q_1 =$	P_1U_1	= 0.4 (108)		= 43
$Q_2 =$	$P_1U_2 + P_2U_1$	= 0.4 (493) +	0.8(108)	= 284
$Q_3 =$	$P_1U_3 + P_2U_2 + P_3U_1$	= 0.4 (601) +	0.8 (493) + 0.6 (108)	= 700
$Q_4 =$	$P_1U_4 + P_2U_3 + P_3U_2$	= 0.4 (565) +	0.8 (601) + 0.6 (493)	= 1,003
$Q_5 =$	$P_1U_5 + P_2U_4 + P_3U_3$	= 0.4 (260) +	0.8 (565) + 0.6 (601)	= 917
$Q_6 =$	$P_1U_6 + P_2U_5 + P_3U_4$	= 0.4 (161) +	0.8 (260) + 0.6(565)	= 611
$Q_7 =$	$P_1U_7 + P_2U_6 + P_3U_5$	= 0.4 (72) +	0.8 (161) + 0.6 (260)	= 314
$Q_8 =$	$P_2U_7 + P_3U_6$	= 0.8 (72) +	0.6 (161)	= 154
$Q_9 =$	P_3U_7	= 0.6 (72)		= 43
				4,069

in which Q_i is the i[th] ordinate of the direct runoff hydrograph, P_i is the i[th] ordinate of the rainfall excess, and U_i is the i[th] ordinate of the unit hydrograph. The sum of the ordinates of the direct runoff hydrograph is 4,069 ft^3/s. Thus, the depth of direct runoff is:

$$\Delta t \sum_{i=1}^{9} Q_i = (0.25\ h)\left(4{,}069\ \frac{ft^3}{s}\right)\left(\frac{3600\ s}{h}\right)\left(\frac{1}{0.88\ mi^2}\right)\left(\frac{1\ mi^2}{27{,}878{,}400\ ft^2}\right)\left(\frac{12\ in}{1\ ft}\right) = 1.8\ in$$

Since the rainfall excess has a depth of 1.8 in, the direct runoff hydrograph must have a depth of 1.8 in. The volume of direct runoff equals the depth of 1.8 in times the drainage area.

6.1.4 Analysis of Unit Hydrographs

Unit hydrographs are either determined from gauged data or they are derived using 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 stream flow gauge 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. A unit hydrograph can be developed at a location where the necessary data are available and then transposed to the design site, as 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 Chapter 3. It would be beneficial to keep a directory of all recording stream gauges 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 a unit hydrograph analysis are rainfall hyetographs and runoff hydrographs for one or more storm events. Ideally, continuous stream flow records for storms that are of a recurrence interval close to the anticipated design recurrence interval would be available. 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 rainfall excess should be isolated and uniform in time and space over the watershed. In addition, the entire basin should be contributing runoff 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, the following procedure is used to derive a unit hydrograph.

6.1.4.1 Base Flow Separation

The first step in developing a unit hydrograph is to plot the measured hydrograph and separate base flow from the total runoff hydrograph. Figure 6.6 illustrates three methods of separating base flow. Each approach describes alternative interactions between stream flow, groundwater, and interflow. These interactions vary from site to site. Prior to the occurrence of the storm, the flow in the stream is from ground-water 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 separated from the total runoff to determine the direct-runoff hydrograph.

The choice of a base flow separation technique should be based on site-specific considerations, including inspection of the behavior of observed total hydrograph data. However, since the base flow is usually small in relation to the flood discharges, the convex separation method (see Figure 6.6c) is adequate for most highway design purposes.

To apply the convex method, two points are identified, the lowest discharge at the start of the rising limb of the hydrograph (point A in Figure 6.6c) and the inflection point on the recession limb (point C in Figure 6.6c). The inflection point occurs at the time when there is a noticeable decrease in the slope of the recession. Starting at point A, a straight line is drawn that has the same slope as that of the hydrograph just prior to the start of the rising limb. The line is extended until the time-to-peak of the hydrograph (point B in Figure 6.6c). A straight line is connected between points B and C. The convex method is applicable where ground-water recharge and possible subsequent increases in base flow 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 locating point C.

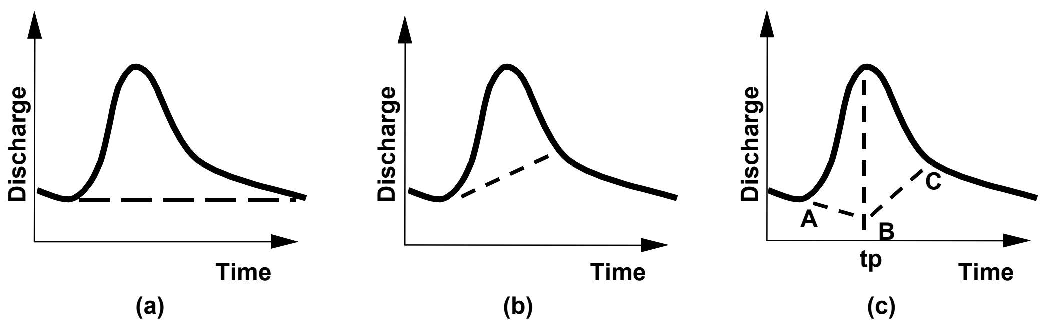


Figure 6.6. Alternative base flow separation methods: (a) constant-discharge; (b) straight-line; and (c) convex methods

6.1.4.2 Determination of the Unit Hydrograph

The direct runoff hydrograph is obtained by subtracting the base flow from the total flood hydrograph. The total volume of direct runoff is the area under the direct runoff hydrograph and can be planimetered, digitized, or computed numerically with either the trapezoidal or Simpson's rule. This area represents a volume of runoff. The volume is next converted to an equivalent depth of rainfall spread uniformly over the entire drainage basin by dividing the volume by the area of the drainage basin. The ordinates of the unit hydrograph are computed by dividing the ordinates of the direct runoff hydrograph by the computed depth of direct runoff. This will yield a unit hydrograph that has a depth of 1 mm (1 in) or a volume of 1 area-mm (area-in), where area is the area of the drainage basin.

6.1.4.3 Estimation of Losses

Losses consist of rainfall that does not contribute to direct runoff. They are the difference between the total rainfall hyetograph and the rainfall-excess hyetograph. Losses can consist of an initial abstraction and losses that occur over the duration of the hyetograph following the start of direct runoff. In many cases, the initial abstraction is considered separately from the other losses. When the computational procedure includes an initial abstraction, it typically consists of all rainfall prior to the start of direct runoff. The remaining losses are separated from the total hyetograph so that the volume of rainfall excess equals the volume of direct runoff.

Any one of several methods can be used to separate losses. The phi-index method is commonly used because of its simplicity. Another method assumes that the losses are a constant proportion of the hyetograph with the proportion set so that the volumes of rainfall excess and direct runoff are equal. The SCS rainfall-runoff equation (Equation 5.21) is also used to separate losses and rainfall excess; this method includes an initial abstraction function defined by Equation 5.20.

By definition, the phi index (φ) equals the average rainfall intensity above which the volume of rainfall excess equals the volume of direct runoff. Thus the value of φ is adjusted so that the volumes of rainfall excess and direct runoff are equal. The procedure for computing the phi index from rainfall and runoff data is:

1. Compute the depths of rainfall (V_p) and direct runoff (V_d).

2. Make an initial estimate of the phi index:

$$\phi = \frac{V_p - V_d}{D} \tag{6.2}$$

   in which D is the duration of rainfall (excluding that part separated as initial abstraction) and φ is an intensity with dimensions of length per unit time.

3. a. Compute the loss function, L(t):

$$L(t) = \begin{cases} \phi & \text{if } \phi \le P(t) \\ P(t) & \text{if } \phi > P(t) \end{cases} \tag{6.3}$$

   where P(t) is the ordinate of the rainfall intensity hyetograph at time t.

   b. Compute the depth of losses, V_L:

$$V_L = \sum_{t=0}^{D} L(t)\,\Delta t \tag{6.4}$$

4. Compute PE(t) = P(t) - L(t) for all ordinates in the rainfall hyetograph (excluding initial abstraction).

5. Compare V_L and V_p - V_d:

   a. If $V_L = V_p - V_d$, go to Step 6.

b. If $V_L < V_p - V_d$, compute the phi-index correction, Δφ:

$$\Delta\phi = \frac{V_p - V_d - V_L}{D_1} \tag{6.5}$$

in which D_1 is the time duration over which PE(t) of step 4 is greater than zero.

c. Adjust the phi index:

$$\phi_{new} = \phi_{old} + \Delta\phi \tag{6.6}$$

d. Return to Step 3.

6. Use the latest value of φ to define losses.

If a large number of storm events are available for analysis, then it may be possible to develop a loss function that can be used in hydrograph synthesis and design. For example, if a phi index is computed for each storm event analyzed, an average phi index may be computed. If values of the phi index are available for numerous watersheds, it may be possible to relate these to soil and/or land cover characteristics. This would enable a loss function to be adopted for an ungauged watershed.

6.1.4.4 Rainfall Excess Hyetograph and Duration

Once the initial abstraction and other losses have been determined, they can be subtracted from the total rainfall hyetograph to determine the rainfall-excess hyetograph. The volume of rainfall excess will equal the volume of direct runoff. The duration of the rainfall excess is especially important because it defines the duration of the corresponding unit hydrograph. For example, if a 5-hour storm produces a 3-hour rainfall-excess hyetograph, a unit hydrograph computed with the corresponding direct runoff hydrograph would be referred to as a 3-hour unit hydrograph.

6.1.4.5 Illustration of the UH Analysis Process

A hypothetical example will be used to illustrate each of the steps of the UH analysis process.

Example 6.2(SI). Figure 6.7(SI) shows a 1-hour rainfall intensity hyetograph. The total volume of rainfall is:

$$P = \sum_{j=1}^{4} i_j \Delta t = \Delta t \sum_{j=1}^{4} i_j = \frac{15 \text{ min}}{60 \text{ min/h}}(6 + 12 + 13 + 3) = 8.5 \text{ mm}$$

The total runoff hydrograph is also shown in Figure 6.7(SI).

The first step is to compute the base flow. The convex method of Section 6.1.4.1 will be used. Since the runoff begins to increase at the start of the second interval, the initial slope of the base flow function will equal the slope in the first 15-minute interval: 0.01 m³/s per 15 minutes. Since the peak of the hydrograph occurs at a storm time of 75 minutes, the initial portion of the base flow function will be extended from a storm time of 15 minutes to a time of 75 minutes.

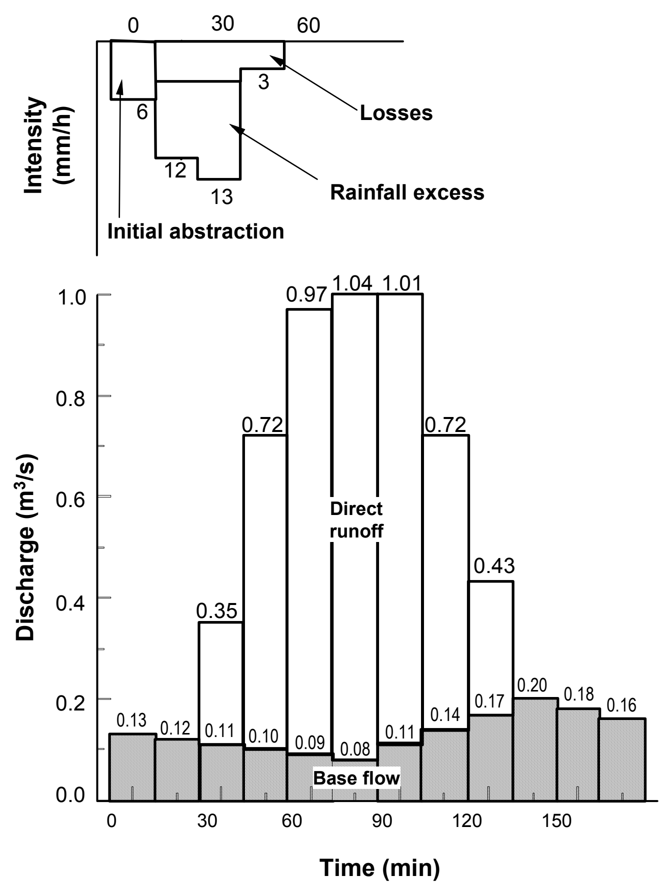


Figure 6.7(SI). Application of unit hydrograph analysis process

Using the decrease of 0.01 m³/s per 15 minutes produces the base flow rates shown in column 3 of Table 6.1. Since there is a noticeable change of slope on the falling limb of the total runoff hydrograph at a storm time of 135 minutes, this will be used as the inflection point; direct runoff will end at a time of 135 minutes. Thus, the second leg of the base flow function can be represented by a linear segment between storm times of 75 and 135 minutes with a slope of:

$$slope = \frac{(0.20 - 0.08)\, m^3/s}{(135 \quad 75)\ min} = 0.002\ m^3/s \text{ per minute}$$

or 0.03 m³/s per 15-minute interval. Because the inflection point has a higher discharge than the base flow at the time to peak, the slope is positive. This slope is used to compute the base flow function for the interval from 75 to 135 minutes. Beyond the inflection point, all of the total runoff is assumed to be base flow. Values for the base flow are given in column 3 of Table 6.1(SI).

Table 6.1(SI). Calculation of Base Flow, Direct Runoff, and Unit Hydrograph

(1) Time (min)	(2) Total Runoff (m^3/s)	(3) Base Flow (m^3/s)	(4) Direct Runoff (m^3/s)	(5) Unit Hydrograph ($m^3/s/mm$)
0	0.13	0.13	0	-
15	0.12	0.12	0	0
30	0.35	0.11	0.24	0.060
45	0.72	0.10	0.62	0.155
60	0.97	0.09	0.88	0.220
75	1.04	0.08	0.96	0.240
90	1.01	0.11	0.90	0.225
105	0.72	0.14	0.58	0.145
120	0.43	0.17	0.26	0.065
135	0.20	0.20	0	0
150	0.18	0.18	0	-
165	0.16	0.16	0	-
				sum = 1.110

The base flow is subtracted from the total runoff to give the direct-runoff hydrograph (column 4 of Table 6.1(SI)). The volume of direct runoff can be computed using the trapezoidal rule:

$$V = \sum_{i=1}^{n} \Delta t \left(\frac{q_i + q_{i+1}}{2} \right) \tag{6.6}$$

where Δt is the time interval, n is the number of ordinates on the direct runoff hydrograph, and q_i are the ordinates of the direct runoff hydrograph. For the values given in column 4 of Table 6.1, the volume is:

$$V = \frac{15\ min(60\ min/h)}{2}[0 + 2(0.24) + 2(0.62) + 2(0.88) + 2(0.96)) + 2(0.90) + 2(0.58) + 2(0.26) + 0]$$
$$= 3996\ m^3$$

If the area of the watershed is 1 km^2, the average depth of direct runoff is:

$$d = \frac{V}{A} = \frac{3{,}996\ m^3 (10^{-6}\ km^2/m^2)(1000\ mm/m)}{1\ km^2} = 4\ mm$$

$$d = \frac{15\,min\,(60\ s/min)(1.110\ m^3/s)(1000\,mm\,/\,m)}{1\ km^2\,(10^6\ m^2/km^2)} = 1\,mm$$

Therefore, the ordinates of the unit hydrograph will be 0.25 times the ordinates of the direct runoff hydrograph (see column 5 of Table 6.1(SI)). The trapezoidal rule can be used to show that the unit hydrograph represents an average depth of 1 mm:

The rainfall intensity hyetograph must be analyzed to find the unit duration of the unit hydrograph. Because the depth of direct runoff, 4 mm, is less than the depth of rainfall, 8.5 mm, losses must be subtracted. Because direct runoff did not begin until the second time interval, all rainfall prior to this (1.5 mm) will be considered an initial abstraction (see column 3 of Table 6.2(SI)). The depth of the remaining rainfall is:

$$\phi = \frac{(7-4)\,mm}{0.75\ h} = 4\ mm\,/\,h$$

in which a time duration of 0.75 hours is used because the first 15-minutes time interval was devoted to initial abstraction. Using Equation 6.3, the losses are 4 mm/h for the second and third time intervals, but only 3 mm/h for the fourth time interval. Thus, the volume of losses is 2.75 mm, which is 0.25 mm less than that necessary to have equal depths of rainfall excess and direct runoff. The phi index can be adjusted using Equation 6.5:

$$\Delta\varphi = \frac{(7-4-2.75)\ mm}{0.5\ h} = 0.5\ \frac{mm}{h}$$

Therefore, Equation 6.6 gives a revised estimate of phi:

$$\varphi_{new} = \varphi_{old} + \Delta\varphi = 4 + 0.5 = 4.5\ mm/h$$

Thus, the new loss function would use 4.5 mm/h when the rainfall exceeds the losses; this is shown in column 5 of Table 6.2(SI). Using the trapezoidal rule, the depth of losses is 3 mm, which is the amount necessary for the depths of direct runoff and rainfall excess to be equal.

The rainfall-excess hyetograph is computed by subtracting both the initial abstraction and the loss function from the rainfall intensity hyetograph. The rainfall excess is given in column 6 of Table 6.2(SI). While the storm event had a duration of 1 hour, the rainfall excess has a duration of 30 minutes. Thus, the unit hydrograph in column 5 of Table 6.1(SI) is defined to be a 30-minute UH. For unit durations other than 30 minutes, the ordinates of the UH would have to be adjusted using the S-hydrograph method.

Table 6.2(SI). Calculation of Phi-Index Loss Function and Rainfall-Excess Hyetograph

(1) Time Interval	(2) Rainfall Intensity (mm/h)	(3) Initial Abstraction (mm/h)	(4) Losses: Trial 1 (mm/h)	(5) Losses: Trial 2 (mm/h)	(6) Rainfall Excess (mm/h)
1	6	6	-	-	0
2	12	0	4	4.5	7.5
3	13	0	4	4.5	8.5
4	3	0	3	3.0	0

Example 6.2(CU). Figure 6.7(CU) shows a 1-hour rainfall intensity hyetograph. The total volume of rainfall is:

$$P = \sum_{j=1}^{4} i_j \Delta t = \Delta t \sum_{j=1}^{4} i_j = \frac{15\ min}{60\ min/h}\left(0.24 + 0.47 + 0.51 + 0.12\right) = 0.335\ in$$

The total runoff hydrograph is also shown in Figure 6.7(CU).

The first step is to compute the base flow. The convex method of Section 6.1.4.1 will be used. Since the runoff begins to increase at the start of the second interval, the initial slope of the base flow function will equal the slope in the first 15-minute interval: 0.4 ft^3/s per 15 minutes. Since the peak of the hydrograph occurs at a storm time of 75 minutes, the initial portion of the base flow function will be extended from a storm time of 15 minutes to a time of 75 minutes. Using the decrease of 0.4 ft^3/s per 15 minutes produces the base flow rates shown in column 3 of Table 6.1(CU). Since there is a noticeable change of slope on the falling limb of the total runoff hydrograph at a storm time of 135 minutes, this will be used as the inflection point; direct runoff will end at a time of 135 minutes. Thus, the second leg of the base flow function can be represented by a linear segment between storm times of 75 and 135 minutes with a slope of:

$$slope = \frac{(7.1 - 2.6)\ ft^3/s}{(135 - 75)\ min} = 0.075\ ft^3/s\ per\ minute$$

or 1.125 ft^3/s per 15-minute interval. Because the inflection point has a higher discharge than the base flow at the time to peak, the slope is positive. This slope is used to compute the base flow function for the interval from 75 to 135 minutes. Beyond the inflection point, all of the total runoff is assumed to be base flow. Values for the base flow are given in column 3 of Table 6.1(CU).

The base flow is subtracted from the total runoff to give the direct-runoff hydrograph (column 4 of Table 6.1(CU)). The volume of direct runoff can be computed using the trapezoidal rule in Equation 6.7:

$$V = \sum_{i=1}^{n} \Delta t \left(\frac{q_i + q_{i+1}}{2}\right) \tag{6.7}$$

where Δt is the time interval, n is the number of ordinates on the direct runoff hydrograph, and q_i are the ordinates of the direct runoff hydrograph. For the values given in column 4 of Table 6.1(CU) the volume is:

$$V = \frac{15\,min\,(60\,min/h)}{2}[0 + 2(8.6) + 2(22.0) + 2(31.3) + 2(34.1) + 2(32.0) + 2(20.6) + 2(9.2) + 0]$$
$$= 142{,}000\,ft^3$$

If the area of the watershed is 0.39 mi², the average depth of direct runoff is:

$$d = \frac{V}{A} = \frac{142{,}000\,ft^3\,(1\ mi^2 / 27{,}878{,}400\,ft^2)(12\ in/ft)}{0.39\,mi^2} = 0.157\ in$$

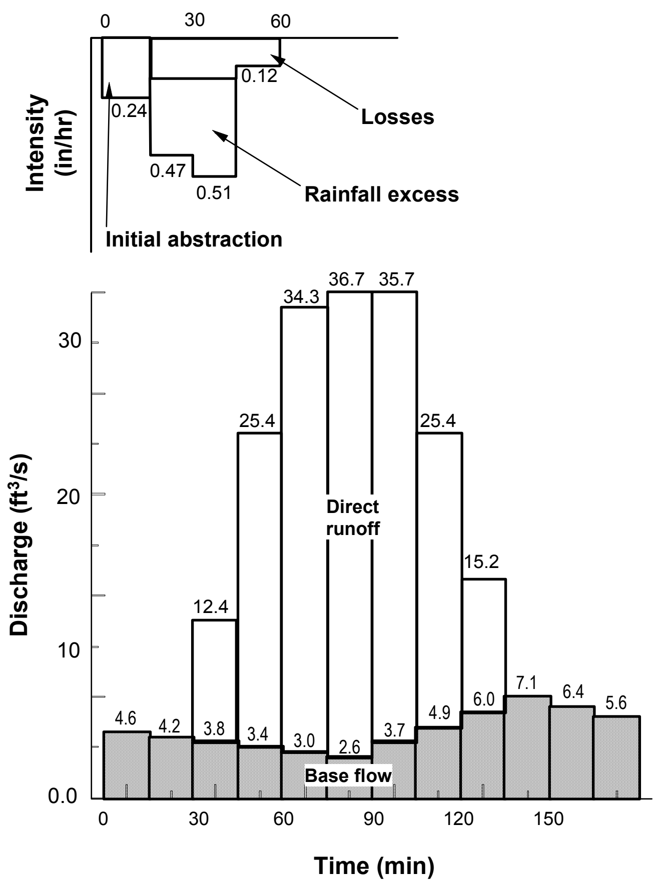


Figure 6.7(CU). Application of unit hydrograph analysis process

Therefore, the ordinates of the unit hydrograph will be 6.37 times (1/0.157) the ordinates of the direct runoff hydrograph (see column 5 of Table 6.1(CU)). The trapezoidal rule can be used to show that the unit hydrograph represents an average depth of 1 in:

$$d = \frac{15\ min(60\ s/min)(1{,}005\ ft^3/s)(12\ in/ft)}{0.39\ mi^2(27{,}878{,}400\ ft^2/mi^2} = 1\ in$$

The rainfall intensity hyetograph must be analyzed to find the unit duration of the unit hydrograph. Because the depth of direct runoff, 0.157 in, is less than the depth of rainfall, 0.335 in, losses must be subtracted. Because direct runoff did not begin until the second time interval, all rainfall prior to this (0.06 in) will be considered an initial abstraction (see column 3 of Table 6.2(CU)). The depth of the remaining rainfall is:

$$\phi = \frac{(0.275 - 0.157)\ in}{0.75\ h} = 0.157\ in/h$$

in which a time duration of 0.75 hour is used because the first 15-minutes time interval was devoted to initial abstraction. Using Equation 6.3, the losses are 0.157 in/h for the second and third time intervals, but only 0.12 in/h for the fourth time interval. Thus, the volume of losses is 0.1085 in, which is 0.0095 in less than that necessary to have equal depths of rainfall excess and direct runoff. The phi index can be adjusted using Equation 6.5:

$$\Delta\phi = \frac{(0.275 - 0.157 - 0.1085)\ in}{0.5\ h} = 0.019\ in/h$$

Therefore, Equation 6.6 gives a revised estimate of phi:

$$\phi_{new} = \phi_{old} + \Delta\phi = 0.157 + 0.019 = 0.176\ in/h$$

Thus, the new loss function would use 0.176 in/h when the rainfall exceeds the losses; this is shown in column 5 of Table 6.2(CU). Using the trapezoidal rule, the depth of losses is 0.118 in, which is the amount necessary for the depths of direct runoff and rainfall excess to be equal.

The rainfall-excess hyetograph is computed by subtracting both the initial abstraction and the loss function from the rainfall intensity hyetograph. The rainfall excess is given in column 6 of Table 6.2(CU). While the storm event had a duration of 1 hour, the rainfall excess has a duration of 30 minutes. Thus, the unit hydrograph in column 5 of Table 6.1(CU) is defined to be a 30-minute UH. For unit durations other than 30 minutes, the ordinates of the UH would have to be adjusted using the S-hydrograph method.

Table 6.1(CU). Calculation of Base Flow, Direct Runoff, and Unit Hydrograph

(1) Time (min)	(2) Total Runoff (ft^3/s)	(3) Base Flow (ft^3/s)	(4) Direct Runoff (ft^3/s)	(5) Unit Hydrograph ($ft^3/s/in$)
0	4.6	4.6	0.0	-
15	4.2	4.2	0.0	0
30	12.4	3.8	8.6	55
45	25.4	3.4	22.0	140
60	34.3	3.0	31.3	199
75	36.7	2.6	34.1	217
90	35.7	3.7	32.0	204
105	25.4	4.9	20.6	131
120	15.2	6.0	9.2	59
135	7.1	7.1	0.0	0
150	6.4	6.4	0.0	
165	5.6	5.6	0.0	
				Sum = 1,005

Table 6.2(CU). Calculation of Phi-Index Loss Function and Rainfall-Excess Hyetograph

(1) Time Interval	(2) Rainfall Intensity (in/h)	(3) Initial Abstraction (in/h)	(4) Losses: Trial 1 (in/h)	(5) Losses: Trial 2 (in/h)	(6) Rainfall Excess (in/h)
1	0.24	0.24			0.000
2	0.47	0.00	0.157	0.176	0.294
3	0.51	0.00	0.157	0.176	0.334
4	0.12	0.00	0.120	0.120	0.000

6.1.5 Derivation of a Unit Hydrograph from a Complex Storm

The method for developing a unit hydrograph given in Section 6.1.4 assumes that the rainfall excess and direct runoff distributions have a simple structure. The convolution process can be reversed with a rainfall-excess hyetograph and a direct-runoff hydrograph to compute a D-hour unit hydrograph for a complex storm.

The analysis procedure consists simply of setting up the equations for computing the n_{ro} ordinates of the direct runoff hydrograph. Since there is only one unknown in the first equation, it can be solved for the first ordinate of the unit hydrograph. The second equation has two unknowns (U_1 and U_2), so the value of U_1 from the solution of the first equation can be used with the second equation to solve for the second ordinate of the unit hydrograph. The process is continued until all of the ordinates have been computed. A problem with this approach is that any round-off error from each computation can accumulate and distort the ordinates of the recession of the unit hydrograph. The problem will be illustrated with an example.

Example 6.3(SI). The direct runoff hydrograph for a 2.33 km^2 watershed given in Table 6.3(SI) is the result of a rainfall excess that consists of three 15-minute periods of equal duration of uniform excess rainfall of 12.4 mm per hour, 7.4 mm per hour, and 2.3 mm per hour. These intensities are equivalent to depths of 3.10, 1.85, and 0.575 mm, respectively, for a total storm excess rainfall of 5.525 mm. If it is assumed that the direct-runoff hydrograph is the composite of three separate hydrographs, each produced by one of the 15-minute periods that have excess rainfall, then it is possible to work backward and derive a 15-minute unit hydrograph that would result in a direct runoff volume of 1 mm.

These calculations are illustrated below and the resulting unit hydrograph is computed in Table 6.5(SI) and plotted in Figure 6.8(SI). The following symbols are used: Q_i = direct runoff hydrograph ordinate (m^3/s), P_i = excess rainfall depth (mm), and U_i = 15-minute unit hydrograph ordinate ($m^3/s/mm$). For each value of the direct runoff hydrograph determined from the gauge data, an equation can be written as shown in column 2 of Table 6.3(SI).

For the first ordinate of the direct runoff hydrograph, only the first ordinate of the unit hydrograph is used. Thus, the solution of the equation ($0.17 = 3.1\ U_1$) yields $U_1 = 0.055$ (see column 4 of Table 6.3(SI)).

Table 6.3(SI). Derivation of Unit Hydrograph from a Complex Storm

(1) Direct Runoff Hydrograph (m^3/s)	(2) Convolution Equation	(3) Equations for Application	(4) Initial Unit Hydrograph ($m^3/s/mm$)	(5) Final Unit Hydrograph ($m^3/s/mm$)
$Q_1 = 0.17$	$= P_1U_1$	$= 3.1U_1$	0.055	0.056
$Q_2 = 0.51$	$= P_1U_2 + P_2U_1$	$=3.1U_2 + 1.85U_1$	0.132	0.134
$Q_3 = 0.91$	$= P_1U_3 + P_2U_2 + P_3U_1$	$= 3.1U_3 + 1.85U_2 + 0.575U_1$	0.205	0.208
$Q_4 = 1.25$	$= P_1U_4 + P_2U_3 + P_3U_2$	$=3.1U_4 + 1.85U_3 + 0.575U_2$	0.257	0.260
$Q_5 = 1.53$	$= P_1U_5 + P_2U_4 + P_3U_3$	$=3.1U_5 + 1.85U_4 + 0.575U_3$	0.302	0.307
$Q_6 = 1.70$	$= P_1U_6 + P_2U_5 + P_3U_4$	$= 3.1U_6 + 1.85U_5 + 0.575U_4$	0.320	0.325
$Q_7 = 1.67$	$= P_1U_7 + P_2U_6 + P_3U_5$	$=3.1U_7 + 1.85U_6 + 0.575U_5$	0.291	0.296
$Q_8 = 1.50$	$= P_1U_8 + P_2U_7 + P_3U_6$	$= 3.1U_8 + 1.85U_7 + 0.575U_6$	0.251	0.254
$Q_9 = 1.28$	$= P_1U_9 + P_2U_8 + P_3U_7$	$=3.1U_9 + 1.85U_8 + 0.575U_7$	0.209	0.212
$Q_{10} = 1.08$	$= P_1U_{10} + P_2U_9 + P_3U_8$	$=3.1U_{10} + 1.85U_9 + 0.575U_8$	0.177	0.180
$Q_{11} = 0.85$	$= P_1U_{11} + P_2U_{10} + P_3U_9$	$=3.1U_{11} + 1.85U_{10} + 0.575U_9$	0.130	0.132
$Q_{12} = 0.65$	$= P_1U_{12} + P_2U_{11} + P_3U_{10}$	$=3.1U_{12} + 1.85U_{11} + 0.575U_{10}$	0.099	0.101
$Q_{13} = 0.51$	$= P_1U_{13} + P_2U_{12} + P_3U_{11}$	$=3.1U_{13} + 1.85U_{12} + 0.575U_{11}$	0.081	0.082
$Q_{14} = 0.34$	$= P_1U_{14} + P_2U_{13} + P_3U_{12}$	$= 3.1U_{14} + 1.85U_{13} + 0.575U_{12}$	0.043	0.043
$Q_{15} = 0.26$	$= P_2U_{14} + P_3U_{13}$	$= + 1.85U_{14} + 0.575U_{13}$		
$Q_{16} = 0.09$	$= P_3U_{14}$	$= 0.575U_{14}$		
			Sum = 2.552	Sum = 2.59

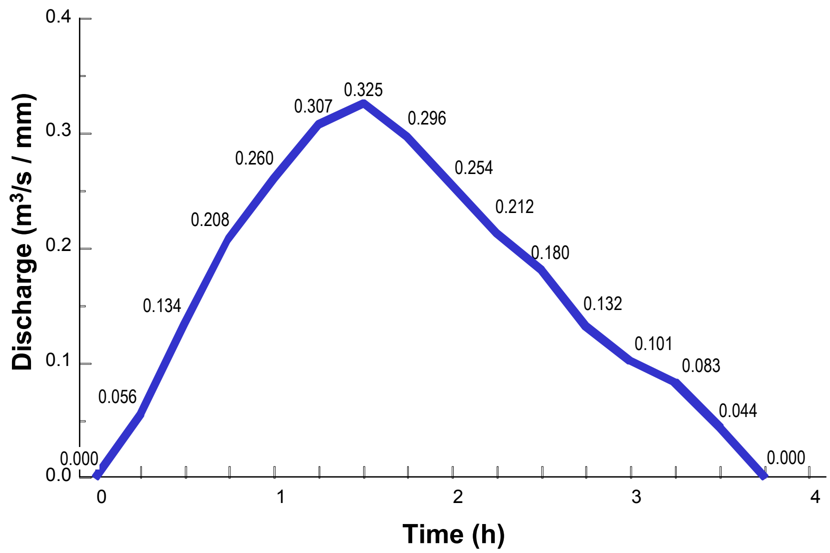


Figure 6.8(SI). Unit hydrograph from Table 6.3(SI)

In the equation for Q_2, the value of U_1 from the solution of the first equation is used so that a value can be computed for U_2:

$$U_2 = \frac{1}{3.1}(0.51 - 1.85\ U_1) = \frac{1}{3.1}[0.51 - 1.85(0.055)] = 0.132\ m^3/s/mm$$

A value can be computed for U_3 using the equation for Q_3 and the computed values of U_1 and U_2:

$$U_3 = \frac{1}{3.1}[0.91 - 1.85(0.132) - 0.575(0.055)] = 0.205\ m^3 / s / mm$$

The process is repeated for U_4 through U_{14} as shown in Table 6.3(SI).

Because of the potential for rounding errors, and to verify that the unit hydrograph represents 1 mm of runoff, the trapezoidal rule is used to verify runoff depth. The sum of the ordinates, as shown in Table 6.3(SI) is 2.552 m^3/s/mm.

$$\Delta t\ (1mm)\sum U_i = 15\,min\,(1mm)\left(2.552\ \frac{m^3}{s \cdot mm}\right)\left(\frac{60\ s}{min}\right)\left(\frac{1000\ mm}{1\ m}\right)\left(\frac{1}{2.33\ km^2}\right)\left(\frac{1\ km}{1000\ m}\right)^2 = 0.986\ mm$$

Rounding errors have resulted in a unit hydrograph that is 1.4% below the required volume. Therefore, all ordinates in column 4 of Table 6.3(SI) are increased by this amount to ensure the proper volume balance. The result is found in column 5., which represents the 15-minute unit hydrograph for this watershed.

Example 6.3(CU). The direct runoff hydrograph for a 0.9 mi^2 watershed given in Table 6.3(CU). It is the result of a rainfall excess that consists of three 15-minute periods of equal duration of uniform excess rainfall of 0.49, 0.29, and 0.9 in/h. These intensities are equivalent to depths of 0.1225, 0.0725, and 0.0225 in, respectively, for a total storm excess rainfall of 0.2175 in. If it is assumed that the direct-runoff hydrograph is the composite of three separate hydrographs, each produced by one of the 15-minute periods that have excess rainfall, it is possible to work backward and derive a 15-minute unit hydrograph that would result in a direct runoff volume of 1 in.

These calculations are illustrated below and the resulting unit hydrograph is computed in Table 6.3(CU) and plotted in Figure 6.8(CU). The following symbols are used: Q_i = direct runoff hydrograph ordinate (ft^3/s), P_i = excess rainfall depth (in), and U_i = 15-minute unit hydrograph ordinate (ft^3/s/in). For each value of the direct runoff hydrograph determined from the gauge data, an equation can be written as shown in column 2 of Table 6.3(CU).

For the first ordinate of the direct runoff hydrograph, only the first ordinate of the unit hydrograph is used. Thus, the solution of the equation ($6=0.1225\ U_1$) yields U_1 = 49 (see column 4 of Table 6.3(CU)).

In the equation for Q_2, the value of U_1 from the solution of the first equation is used so that a value can be computed for U_2:

$$U_2 = \frac{1}{0.1225}\left(18 - 0.0725\ U_1\right) = \frac{1}{0.1225}\left[18 - 0.0725(49)\right] = 118\, ft^3/s/in$$

A value can be computed for U_3 using the equation for Q_3 and the computed values of U_1 and U_2:

$$U_3 = \frac{1}{0.1225}\left[32 - 0.0725(118) - 0.0225(49)\right] = 182\, ft^3/s/in$$

The process is repeated for U_4 through U_{14} as shown in Table 6.3(CU).

Because of the potential for rounding errors, and to verify that the unit hydrograph represents 1 in of runoff, the trapezoidal rule is used to verify volume. The sum of the ordinates, as shown in Table 6.3(CU) is 2,286 ft^3/s/in.

$$\Delta t\,(1\ in)\sum U_i = 15\ min\,(1\ in)\left(2{,}286\ \frac{ft^3}{s\ in}\right)\left(\frac{60\ s}{min}\right)\left(\frac{12\ in}{1\ ft}\right)\left(\frac{1}{0.9\ mi^2}\right)\left(\frac{1\ mi}{5280\ ft}\right)^2 = 0.984\ in$$

Rounding errors have resulted in a unit hydrograph that is 1.6% below the required volume. Therefore, all ordinates in column 4 are increased by this amount to insure the proper volume balance. The result is found in column 5, which represents the 15-minute unit hydrograph for this watershed.

Table 6.3(CU). Derivation of Unit Hydrograph from a Complex Storm

(1) Direct Runoff Hydrograph (ft^3/s)	(2) Convolution Equations	(3) Equations for Application	(4) Initial Unit Hydrograph ($ft^3/s/in$)	(5) Final Unit Hydrograph ($ft^3/s/in$)
Q_1 = 6	$= P_1U_1$	$= 0.49U_1$	49	50
Q_2 = 18	$= P_1U_2 + P_2U_1$	$=0.49U_2 + 0.29U_1$	118	120
Q_3 = 32	$= P_1U_3 + P_2U_2 + P_3U_1$	$= 0.49U_3 + 0.29U_2 + 0.09U_1$	182	185
Q_4 = 44	$= P_1U_4 + P_2U_3 + P_3U_2$	$=0.49U_4 + 0.29U_3 + 0.09U_2$	230	233
Q_5 =54	$= P_1U_5 + P_2U_4 + P_3U_3$	$=0.49U_5 + 0.29U_4 + 0.09U_3$	271	276
Q_6 = 60	$= P_1U_6 + P_2U_5 + P_3U_4$	$= 0.49U_6 + 0.29U_5 + 0.09U_4$	287	292
Q_7 = 59	$= P_1U_7 + P_2U_6 + P_3U_5$	$=0.49U_7 + 0.29U_6 + 0.09U_5$	262	266
Q_8 = 53	$= P_1U_8 + P_2U_7 + P_3U_6$	$= 0.49U_8 + 0.29U_7 + 0.09U_6$	225	229
Q_9 = 45	$= P_1U_9 + P_2U_8 + P_3U_7$	$=0.49U_9 + 0.29U_8 + 0.09U_7$	186	189
Q_{10} = 38	$= P_1U_{10} + P_2U_9 + P_3U_8$	$=0.49U_{10} + 0.29U_9 + 0.09U_8$	159	161
Q_{11} = 30	$= P_1U_{11} + P_2U_{10} + P_3U_9$	$=0.49U_{11} + 0.29U_{10} + 0.09U_9$	117	119
Q_{12} = 23	$= P_1U_{12} + P_2U_{11} + P_3U_{10}$	$=0.49U_{12} + 0.29U_{11} + 0.09U_{10}$	89	91
Q_{13} = 18	$= P_1U_{13} + P_2U_{12} + P_3U_{11}$	$=0.49U_{13} + 0.29U_{12} + 0.09U_{11}$	73	74
Q_{14} = 12	$= P_1U_{14} + P_2U_{13} + P_3U_{12}$	$= 0.49U_{14} + 0.29U_{13} + 0.09U_{12}$	39	39
Q_{15} = 9	$= P_2U_{14} + P_3U_{13}$	$= + 0.29U_{14} + 0.09U_{13}$		
Q_{16} = 3	$= P_3U_{14}$	$= 0.09U_{14}$		
			Sum = 2,287	Sum = 2,324

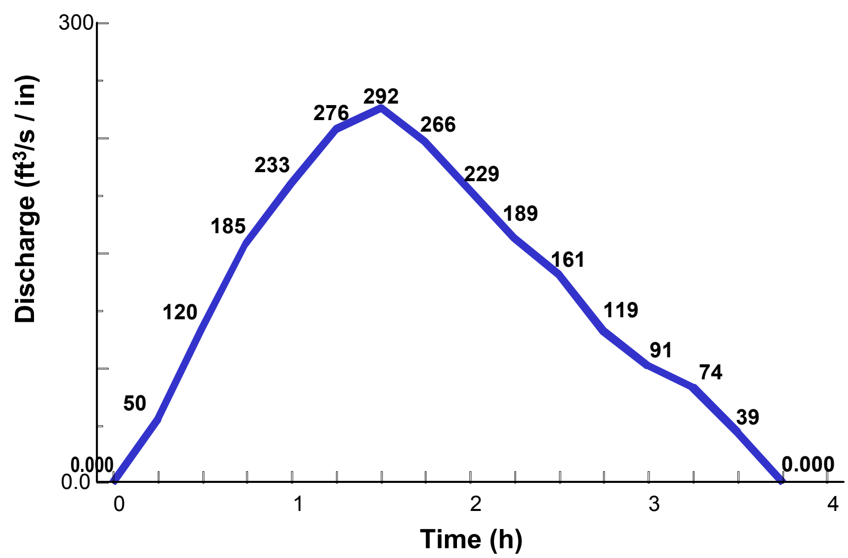


Figure 6.8(CU). Unit hydrograph from Table 6.3(CU)

6.1.6 Averaging Storm-Event Unit Hydrographs

Unit hydrographs analyzed from different storm events on the same watershed will have widely different shapes even if the durations of rainfall excess are similar. These differences are illustrated in Figure 6.9 and can be due to differences in storm patterns, storm volumes, storm-cell movement, and antecedent watershed conditions.

The following steps are used to average two or more unit hydrographs computed from different storm events on the same watershed:

1. Compute the average peak discharge of the unit hydrographs;
2. Compute the average time to peak;
3. Plot each of the storm-event UH's on a single graph;
4. Locate the point defined by the average peak discharge and average time to peak from Steps 1 and 2;
5. Sketch a unit hydrograph that represents an average of the shapes of the storm-event UH's and passes through the point defined in Step 4;
6. Read off the ordinates of the average unit hydrograph sketched in Step 5 and compute the volume of the average UH; and
7. Adjust the ordinates of the sketched UH so that it has a volume of 1 area-mm (area-in); the adjustments are usually made in the recession of the UH.

This averaging method assumes that all of the storm-event UHs have approximately the same unit duration. If they do not, they should be adjusted using the S-hydrograph method (see Section 6.3.1) prior to averaging.

It is important to emphasize that it is incorrect to compute an average of the ordinates for each time. If this were done, the watershed-average UH would have a low peak discharge and the shape would not be representative of the true unit hydrograph.

Example 6.4. Figure 6.9 shows unit hydrographs for five storm events on White Oak Bayou, TX; the data were adapted from graphs provided by Hare (1970). The storm dates were:

1. January 31 – February 6, 1952
2. August 28 – September 3, 1953
3. February 3 – February 10, 1955
4. February 1 – February 2, 1959
5. June 26 – June 28, 1960

The ordinates are given in Table 6.4. White Oak Bayou has a drainage area of 238 km^2 (92 mi^2). The peak discharge and time to peak are given in Table 6.4 for each storm UH; the averages are also given. The point defined by $\overline{q}_p$ and $\overline{t}_p$ is located on Figure 6.9 with the five storm-event UHs. A smooth distribution was sketched through the point, with consideration given to the shapes of the five storm-event unit hydrographs. The ordinates at 2-hour intervals were taken from the initial sketch of the average UH and the volume under the curve computed using the trapezoidal rule. The ordinates were adjusted because the volume of the initially sketched UH was greater than 1 area-mm (1 area-in). The sum of the ordinates for the final UH are shown in Table 6.4. The depth is:

$$depth = 32.18 \frac{m^3}{s \cdot mm}(1\ mm)(2\ h)\left(\frac{3600\ s}{h}\right)\left(\frac{1000\ mm}{1\ m}\right)\left(\frac{1}{238\ km^2}\right)\left(\frac{1\ km}{1000\ m}\right)^2 = 0.97\ mm \quad \text{(SI)}$$

$$depth = 28,870 \frac{ft^3}{s \cdot in}(1\ in)(2\ h)\left(\frac{3600\ s}{h}\right)\left(\frac{12\ in}{1\ ft}\right)\left(\frac{1}{92\ mi^2}\right)\left(\frac{1\ mi}{5280\ ft}\right)^2 = 0.97\ in \quad \text{(CU)}$$

The difference of 0.03 mm (0.03 in) is assumed to be the volume in the recession of the UH beyond a storm time of 60 hours.

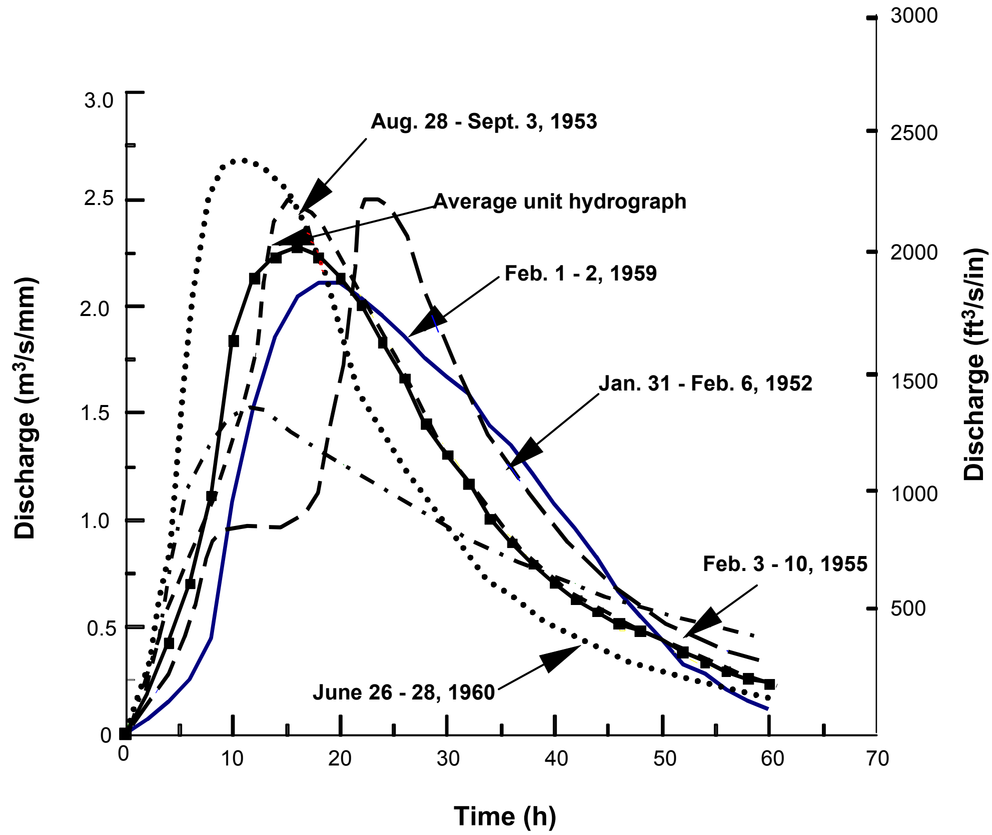


Figure 6.9. Observed unit hydrographs, White Oak Bayou

Table 6.4(SI). Computing a Watershed Unit Hydrograph from Five Storm-Event Unit Hydrographs, White Oak, Bayou, TX

Time (h)	Discharge (m^3/s/mm) for UH					Average UH (m^3/s/mm)	Dimensionless UH	
	1952	1953	1955	1959	1960		q/q_p	t/t_p
0	0.000	0.000	0.000	0.000	0.000	0.000	0	0
2	0.067	0.212	0.234	0.111	0.312	0.189	0.083	0.125
4	0.145	0.580	0.758	0.268	0.892	0.424	0.186	0.250
6	0.251	0.791	1.182	0.502	1.973	0.702	0.309	0.375
8	0.446	1.059	1.360	0.914	2.564	1.115	0.490	0.500
10	1.081	1.349	1.471	0.981	2.675	1.839	0.809	0.625
12	1.527	1.694	1.527	0.970	2.675	2.129	0.936	0.750
14	1.850	2.486	1.460	0.925	2.597	2.229	0.980	0.875
16	2.040	2.475	1.371	1.003	2.452	2.274	1.000	1.000
18	2.107	2.419	1.326	1.126	2.240	2.229	0.980	1.125
20	2.107	2.240	1.271	1.639	1.895	2.129	0.936	1.250
22	2.034	2.040	1.204	2.508	1.561	2.006	0.882	1.375
24	1.951	1.873	1.148	2.508	1.416	1.828	0.804	1.500
26	1.862	1.683	1.092	2.341	1.237	1.661	0.730	1.625
28	1.750	1.471	1.025	2.018	1.092	1.449	0.637	1.750
30	1.672	1.326	0.981	1.795	0.981	1.304	0.574	1.875
32	1.583	1.193	0.925	1.583	0.836	1.170	0.515	2.000
34	1.438	1.025	0.869	1.349	0.725	1.003	0.441	2.125
36	1.349	0.914	0.825	1.237	0.669	0.892	0.392	2.250
38	1.215	0.803	0.780	1.092	0.568	0.791	0.348	2.375
40	1.070	0.702	0.725	0.947	0.513	0.702	0.309	2.500
42	0.959	0.624	0.702	0.858	0.457	0.624	0.275	2.625
44	0.825	0.557	0.669	0.758	0.412	0.568	0.250	2.750
46	0.669	0.490	0.635	0.669	0.368	0.513	0.225	2.875
48	0.546	0.457	0.602	0.602	0.334	0.479	0.211	3.000
50	0.435	0.412	0.568	0.535	0.301	0.435	0.191	3.125
52	0.323	0.357	0.535	0.468	0.268	0.379	0.167	3.250
54	0.279	0.312	0.513	0.435	0.245	0.334	0.147	3.375
56	0.201	0.279	0.490	0.390	0.223	0.290	0.127	3.500
58	0.156	0.245	0.468	0.357	0.201	0.256	0.113	3.625
60	0.111	0.223	0.457	0.334	0.178	0.234	0.103	3.750
Sum	32.047	32.292	27.176	31.222	32.861	32.181		
Depth (mm)	0.97	0.98	0.82	0.94	0.99	0.97		
Peak (m^3/s/mm)	2.107	2.486	1.527	2.51	2.675			
Time to Peak (h)	19.0	14.8	11.4	23.4	10.9			

Table 6.4(CU). Computing a Watershed Unit Hydrograph from Five Storm-Event Unit Hydrographs, White Oak, Bayou, TX

Time (h)	Discharge (ft^3/s/in) for UH					Average UH (ft^3/s/in)	Dimensionless UH	
	1952	1953	1955	1959	1960		q/q_p	t/t_p
0	0	0	0	0	0	0	0.000	0.000
2	60	190	210	100	280	170	0.083	0.125
4	130	520	680	240	800	380	0.186	0.250
6	225	709	1060	450	1770	630	0.309	0.375
8	400	950	1220	820	2300	1000	0.490	0.500
10	970	1210	1320	880	2400	1650	0.809	0.625
12	1370	1520	1370	870	2400	1910	0.936	0.750
14	1660	2230	1310	830	2330	2000	0.980	0.875
16	1830	2220	1230	900	2200	2040	1.000	1.000
18	1890	2170	1190	1010	2010	2000	0.980	1.125
20	1890	2010	1140	1470	1700	1910	0.936	1.250
22	1820	1830	1080	2250	1400	1800	0.882	1.375
24	1750	1680	1030	2250	1270	1640	0.804	1.500
26	1670	1510	980	2100	1110	1490	0.730	1.625
28	1570	1320	920	1810	980	1300	0.637	1.750
30	1500	1190	880	1610	880	1170	0.574	1.875
32	1420	1070	830	1420	750	1050	0.515	2.000
34	1290	920	780	1210	650	900	0.441	2.125
36	1210	820	740	1110	600	800	0.392	2.250
38	1090	720	700	980	510	709	0.348	2.375
40	960	630	650	850	460	630	0.309	2.500
42	860	560	630	770	410	560	0.275	2.625
44	740	500	600	680	370	510	0.250	2.750
46	600	440	570	600	330	460	0.225	2.875
48	490	410	540	540	300	430	0.211	3.000
50	390	370	510	480	270	390	0.191	3.125
52	290	320	480	420	240	340	0.167	3.250
54	250	280	460	390	220	300	0.147	3.375
56	180	250	440	350	200	260	0.127	3.500
58	140	220	420	320	180	230	0.113	3.625
60	100	200	410	300	160	210	0.103	3.750
Sum	28745	28969	24380	28010	29480	28870		
Depth (in)	0.97	0.98	0.82	0.94	0.99	0.97		
Peak (ft^3/s/in)	1890	2230	1390	2280	2400			
Time to peak (h)	19.0	14.8	11.4	23.4	10.9			

6.1.7 Unit Hydrograph Limitations

Because of the assumptions made in the development of unit hydrograph procedures, a designer should be familiar with several limitations and sources of error. Uniformity of rainfall intensity and duration over the drainage basin is a requirement that is seldom met. For this reason it is best to use large storms covering a major portion of the drainage area when developing unit hydrographs. 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, the time base, and the time to peak, with the shape of the final unit hydrograph adjusted to a depth of 1 mm (1 in) of runoff.

The lack of stations with recording rain gauges makes it very difficult to obtain accurate rainfall distribution data. Even bucket-type gauges 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.

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 lack of continuous records or gauge malfunction.

In order to overcome some of these limitations, unit hydrograph development should be limited to drainage areas less than 2,600 km^2 (1,000 mi^2). In addition, when applying the unit hydrograph to a synthetic design storm, the design storm should be sufficiently long to allow the entire watershed to contribute to the outlet point. Since a design storm may not be of uniform intensity, the design storm length should be between 1 and 1.7 times the time of concentration of the watershed. In the case of the SCS 24-hour design storm, this guidance implies that its use may be limited to watersheds with a time of concentration less than 14 to 24 hours.

6.2 DEVELOPMENT OF A DESIGN STORM

The volume, duration, frequency, and intensity of storms have been discussed. Some design problems only require either the volume of rainfall or an average intensity for a specified duration and frequency. For example, the rational method uses the rainfall intensity for a specified return period. However, for many problems in hydrologic design, it is necessary to show the variation of the rainfall volume with time. That is, some hydrologic design problems require the storm input to the design method to be expressed as a hyetograph and not just as a total volume for the storm. Characteristics of a hyetograph that are important are the peak, the time to peak, the distribution, and the volume, duration, and frequency. Design methods most often use a synthetic design storm rather than an actual storm hyetograph.

In developing a design storm hyetograph for any region, empirical analyses of measured rainfall records are made to determine the most likely arrangement of the ordinates of the hyetograph. Some storm events will have an early peak (i.e., front loaded), some a late peak (i.e., rear loaded), some will peak in the center of the storm (i.e., center loaded), and some will have more than one peak. The empirical analysis of measured rainfall hyetographs at a location will show the most likely of these possibilities, and this finding can be used to develop the design storm.

6.2.1 Constant-Intensity Design Storm

A design storm that is used frequently for hydrologic designs on very small urban watersheds is the constant-intensity storm. It is quite common to assume that the critical cause of flooding is the short-duration, high-intensity storm. In most cases, it has been shown that the largest peak runoff rate occurs when the entire drainage area is contributing and so, it is common to set the duration of the design storm equal to the time of concentration of the watershed. The intensity of the storm is obtained from an intensity-duration-frequency curve for the location, using the frequency specified by the design standard; the storm depth is the intensity multiplied by the time of concentration.

Example 6.5. To illustrate the constant-intensity design storm, assume the following conditions: (I) the design standard specifies a 10-year return period for design; (2) the watershed time of concentration is 15 minutes; and (3) the watershed is located in Baltimore, Maryland. The rainfall intensity for a 10-year return period and a duration of 15 minutes is 140 mm/h (5.5 in/h), which yields a storm depth of 35 mm (1.375 in). The resulting design storm is shown in Figure 6.10.

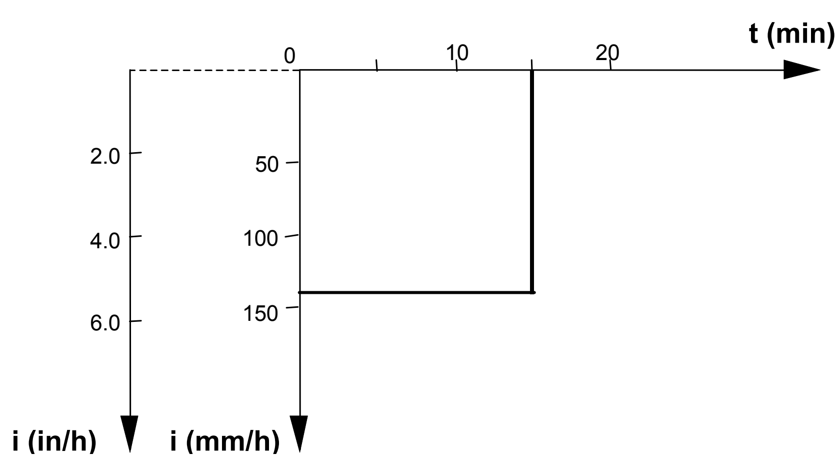


Figure 6.10. Constant-intensity design storm for a 15-minute time of concentration and a 10-year return period, Baltimore, MD

6.2.2 The SCS 24-Hour Storm Distributions

The SCS developed four dimensionless rainfall distributions using the Weather Bureau's Rainfall Frequency Atlases. The rainfall frequency data for areas less than 1050 km^2 (405 mi^2) for durations to 24 hours, and for frequencies from 1 to 100 years were used. Data analyses indicated four major regions, and the resulting rainfall distributions were labeled type I, IA, II, or III. The locations where these design storms should be used are shown in Figure 6.11.

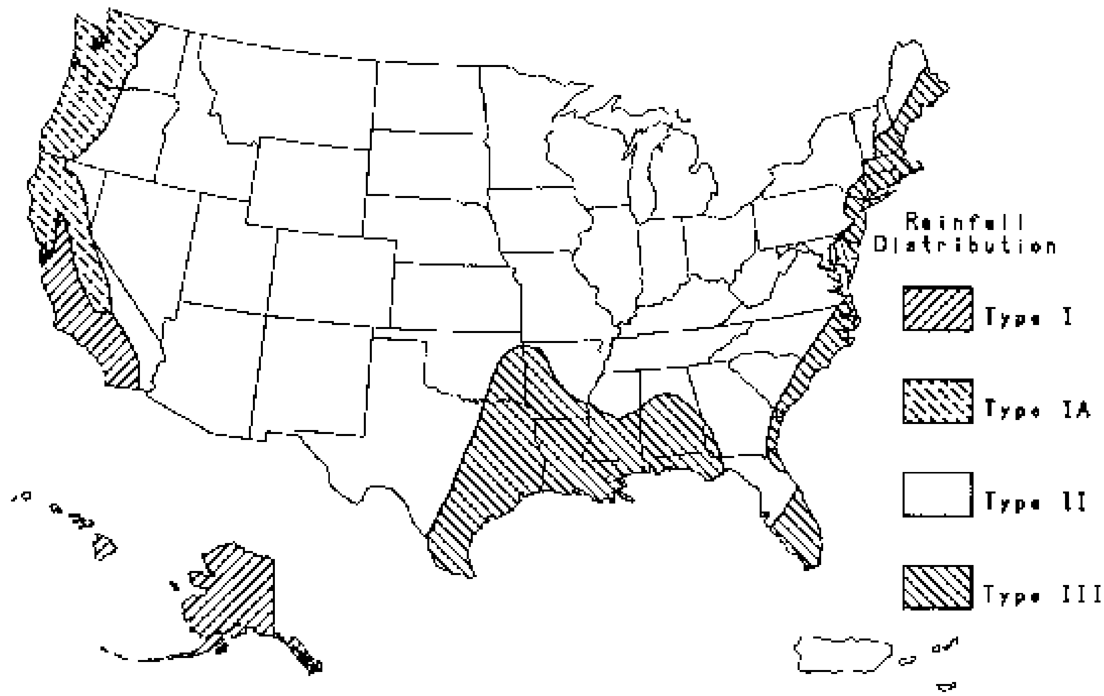


Figure 6.11. Approximate geographic areas for SCS rainfall distribution

The distributions are based on the generalized rainfall volume-duration-frequency relationships shown in technical publications of the Weather Bureau. Rainfall depths for durations from 6 minutes to 24 hours were obtained from the volume-duration-frequency information in these publications and used to derive the storm distributions. Using increments of 6 minutes, incremental rainfall depths were determined (see Figure 6.12). For example, the maximum 6-minute depth was subtracted from the maximum 12-minute depth and this 12-minute depth was subtracted from the maximum 18-minute depth, and so on to 24 hours. The distributions were formed by arranging these 6-minute incremental depths such that for any duration from 6 minutes to 24 hours, the rainfall depth for that duration and frequency will be represented as a continuous sequence of 6-minute depths.

The location of the peak was found from the analysis of measured storm events to be location dependent. For the regions with type I and IA storms, the peak intensity occurred at a storm time of about 8 hours, while for the regions with type II and III storms, the peak was found to occur at the center of the storm. Therefore for type II and III storm events, the greatest 6-minute depth is assumed to occur at about the middle of the 24-hour period, the second largest 6-minute incremental depth in the next 6 minutes, and the third largest in the 6-minute interval preceding the maximum 6-minute depth. This continues with each incremental rainfall depth to be of decreasing order of magnitude. Thus the smaller increments fall at the beginning and end of the 24-hour storm.

This procedure results in the maximum 6-minute depth being contained within the maximum I-hour depth, the maximum I-hour depth is contained within the maximum 6-hour depth, and so on. Because all of the critical storm depths are contained within the storm distributions, the distributions are appropriate for designs on both small and large watersheds.

The resulting distributions (Figure 6.12) are most often presented with the ordinates given on a dimensionless scale. The SCS type I and type II dimensionless distributions plot as a straight line on log-log paper. Although they do not agree exactly with IDF values from all locations in the region for which they are intended, the differences are within the accuracy of the rainfall depths read from the Weather Bureau atlases. The ordinates are given in Table 6.5 and shown in Figure 6.12 for the four SCS synthetic design storms.

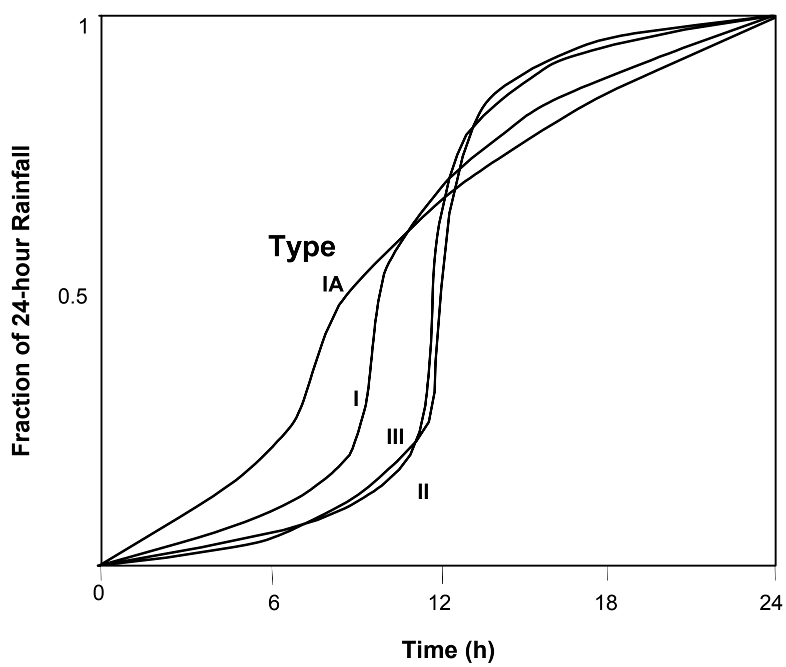


Figure 6.12. SCS 24-hour rainfall distributions (not to scale)

Table 6.5. SCS Cumulative Dimensionless 24-hour Storms

Time (h)	Type I Storm	Type IA Storm	Type II Storm	Type III Storm
0	0	0	0	0
0.5	0.008	0.010	0.005	0.005
1.0	0.017	0.020	0.011	0.010
1.5	0.026	0.035	0.016	0.015
2.0	0.035	0.050	0.022	0.020
2.5	0.045	0.067	0.028	0.025
3.0	0.055	0.082	0.035	0.031
3.5	0.065	0.098	0.041	0.037
4.0	0.076	0.116	0.048	0.043
4.5	0.087	0.135	0.056	0.050
5.0	0.099	0.156	0.063	0.057
5.5	0.112	0.180	0.071	0.064
6.0	0.126	0.206	0.080	0.072
6.5	0.140	0.237	0.089	0.081
7.0	0.156	0.268	0.098	0.091
7.5	0.174	0.310	0.109	0.102
8.0	0.194	0.425	0.120	0.114
8.5	0.219	0.480	0.133	0.128
9.0	0.254	0.520	0.147	0.146
9.5	0.303	0.550	0.162	0.166
10.0	0.515	0.577	0.181	0.189
10.5	0.583	0.601	0.204	0.212
11.0	0.624	0.624	0.235	0.250
11.5	0.655	0.645	0.283	0.298
12.0	0.682	0.664	0.663	0.500
12.5	0.706	0.683	0.735	0.702
13.0	0.728	0.701	0.772	0.750
13.5	0.748	0.719	0.799	0.784
14.0	0.766	0.736	0.820	0.811
14.5	0.783	0.753	0.838	0.834
15.0	0.799	0.769	0.854	0.854
15.5	0.815	0.785	0.868	0.872
16.0	0.830	0.800	0.880	0.886
16.5	0.844	0.815	0.891	0.898
17.0	0.857	0.830	0.902	0.910
17.5	0.870	0.844	0.912	0.920
18.0	0.882	0.858	0.921	0.928
18.5	0.893	0.871	0.929	0.936
19.0	0.905	0.884	0.937	0.943
19.5	0.916	0.896	0.945	0.950
20.0	0.926	0.908	0.952	0.957
20.5	0.936	0.920	0.959	0.963
21.0	0.946	0.932	0.965	0.969
21.5	0.956	0.944	0.972	0.975
22.0	0.965	0.956	0.978	0.981
22.5	0.974	0.967	0.984	0.986
23.0	0.983	0.978	0.989	0.991
23.5	0.992	0.989	0.995	0.996
24.0	1.000	1.000	1.000	1.000

<u>Example 6.6(SI)</u>. The procedure used to form a design storm can be illustrated with a simplified example. The design storm will have the following characteristics: duration, 6 hours; frequency, 50 years; time increment, 1 hour; location, Baltimore. The Baltimore intensity-duration-frequency curve was used to obtain the rainfall intensities for durations of 1 to 6 hours in increments of 1 hour; these intensities are given in column 3 of Table 6.6(SI). The depth (i.e., duration times intensity) is given in column 4, with the incremental depth (column 5) being set equal to the difference between the depths for durations 1 hour apart. A center-loaded storm distribution is assumed. The incremental depths are used to form the 50-year design storm, which is given in column 6, by placing the largest incremental depth in hour 3 and the second largest incremental depth in hour 4; the remaining incremental depths are positioned by alternating their location before and after the maximum incremental depth. The maximum 3 hours of the design storm has a depth of 99 mm that is the depth for a 3-hour duration from the Baltimore IDF curve; this will be true for any storm duration from 1 to 6 hours. The cumulative form of the design storm is given in column 7 of Table 6.6(SI).

A dimensionless design storm can be developed by transforming the cumulative design storm of column 7 of Table 6.6(SI) by dividing it by the total depth of 117.6 mm. The dimensionless cumulative design storm derived from the 50-year intensities is shown in column 8 of Table 6.6(SI). The calculation of a dimensionless design storm for a 2-year frequency is shown in the lower part of Table 6.6(SI).

In comparing the 50-year and 2-year dimensionless design storms, it should be apparent that a cumulative design storm could be developed for any design frequency by multiplying the 6-hour rainfall depth for that frequency by the average ordinates of the dimensionless cumulative design storms of Table 6.6(SI), which is approximately [0.05, 0.14, 0.78, 0.90, 0.97, 1.00]. Based on this dimensionless cumulative design storm (see Figure 6.13(SI)a), the 10-year cumulative design storm, which has a 6-hour depth of 88.2 mm (14.7 mm/h from the Baltimore IDF curve multiplied by 6 hours), would be [4.4, 12.3, 68.8, 79.4, 85.6, 88.2 mm], which is also shown in Figure 6.13(SI)a. Thus the 10-year design storm would be [4.4, 7.9, 56.5, 10.6, 6.2, 2.6 mm/h]. The 10-year, 6-hour design storm is shown in Figure 6.13(SI)b with ordinates expressed as intensities.

Table 6.6(SI). Development of 6-hour Dimensionless Cumulative Design Storms for Baltimore

(1) T (yr)	(2) Duration (h)	(3) Intensity (mm/h)	(4) Depth (mm)	(5) Incre-mental Depth (mm)	(6) Design Storm (mm)	(7) Cumulative Design Storm (mm)	(8) Dimensionless Cumulative Design Storm
50	1	76.2	76.2	76.2	6.2	6.2	0.053
	2	44.5	89.0	12.8	10.0	16.2	0.138
	3	33.0	99.0	10.0	76.2	92.4	0.786
	4	26.7	106.8	7.8	12.8	105.2	0.895
	5	22.6	113.0	6.2	7.8	113.0	0.961
	6	19.6	117.6	4.6	4.6	117.6	1.000
2	1	34.3	34.3	34.3	2.7	2.7	0.049
	2	20.8	41.6	7.3	4.9	7.6	0.139
	3	15.5	46.5	4.9	34.3	41.9	0.767
	4	12.7	50.8	4.3	7.3	49.2	0.901
	5	10.7	53.5	2.7	4.3	53.5	0.980
	6	9.1	54.6	1.1	1.1	54.6	1.000

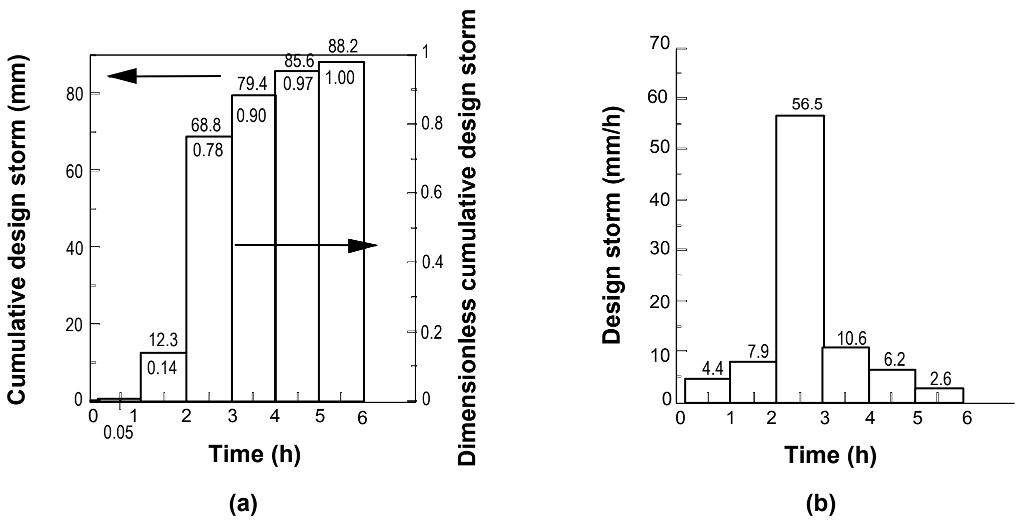


Figure 6.13(SI). 6-hour design storms for Example 6.6(SI): (a) dimensionless cumulative design storm and the cumulative design storm and (b) the design storm

Example 6.6(CU). The procedure used to form a design storm can be illustrated with a simplified example. The design storm will have the following characteristics: duration, 6 hours; frequency, 50 years; time increment, 1 hour; location, Baltimore. The Baltimore intensity-duration-frequency curve was used to obtain the rainfall intensities for durations of 1 to 6 hours in increments of 1 hour; these intensities are given in column 3 of Table 6.6(CU). The depth (i.e., duration times intensity) is given in column 4, with the incremental depth (column 5) being set equal to the difference between the depths for durations 1 hour apart. A center-loaded storm distribution is assumed. The incremental depths are used to form the 50-year design storm, which is given in column 6, by placing the largest incremental depth in hour 3 and the second largest incremental depth in hour 4; the remaining incremental depths are positioned by alternating their location before and after the maximum incremental depth. The maximum 3 hours of the design storm has a depth of 3.9 in which is the depth for a 3-hour duration from the Baltimore IDF curve; this will be true for any storm duration from 1 to 6 hours. The cumulative form of the design storm is given in column 7 of Table 6.6(CU).

A dimensionless design storm can be developed by transforming the cumulative design storm of column 7 of Table 6.6(CU) by dividing it by the total depth of 4.62 in. The dimensionless cumulative design storm derived from the 50-year intensities is shown in column 8 of Table 6.6(CU). The calculation of a dimensionless design storm for a 2-year frequency is shown in the lower part of Table 6.6(CU).

In comparing the 50-year and 2-year dimensionless design storms, it should be apparent that a cumulative design storm could be developed for any design frequency by multiplying the 6-hour rainfall depth for that frequency by the average ordinates of the dimensionless cumulative design storms of Table 6.6(CU), which is approximately [0.05, 0.14, 0.78, 0.90, 0.97, 1.00]. Based on this dimensionless cumulative design storm (see Figure 6.13(CU) (a)), the 10-year cumulative design storm, which has a 6-hour depth of 3.48 in (0.58 in/h) from the Baltimore IDF curve multiplied by 6 hours), would be [0.17, 0.49, 2.71, 3.13, 3.38, 3.48 in], which is also shown in Figure 6.13(CU). Thus the 10-year design storm would be [0.17, 0.32, 2.22, 0.42, 0.25, 0.10 in/h]. The 10-year, 6-hour design storm is shown in Figure 6.13(CU)(b) with ordinates expressed as intensities.

Table 6.6(CU). Development of 6-hour Dimensionless Cumulative Design Storms for Baltimore

(1) T (yr)	(2) Duration (h)	(3) Intensity (in/h)	(4) Depth (in)	(5) Incre-mental Depth (in)	(6) Design Storm (in)	(7) Cumulative Design Storm (in)	(8) Dimensionless Cumulative Design Storm
50	1	3.00	3.00	3.00	0.25	0.25	0.05
	2	1.75	3.50	0.50	0.40	0.65	0.14
	3	1.30	3.90	0.40	3.00	3.65	0.79
	4	1.05	4.20	0.30	0.50	4.15	0.90
	5	0.89	4.45	0.25	0.30	4.45	0.96
	6	0.77	4.62	0.17	0.17	4.62	1.00
2	1	1.35	1.35	1.35	0.10	0.10	0.05
	2	0.82	1.64	0.29	0.19	0.29	0.13
	3	0.61	1.83	0.19	1.35	1.64	0.76
	4	0.50	2.00	0.17	0.29	1.93	0.89
	5	0.42	2.10	0.10	0.17	2.10	0.97
	6	0.36	2.16	0.06	0.06	2.16	1.00

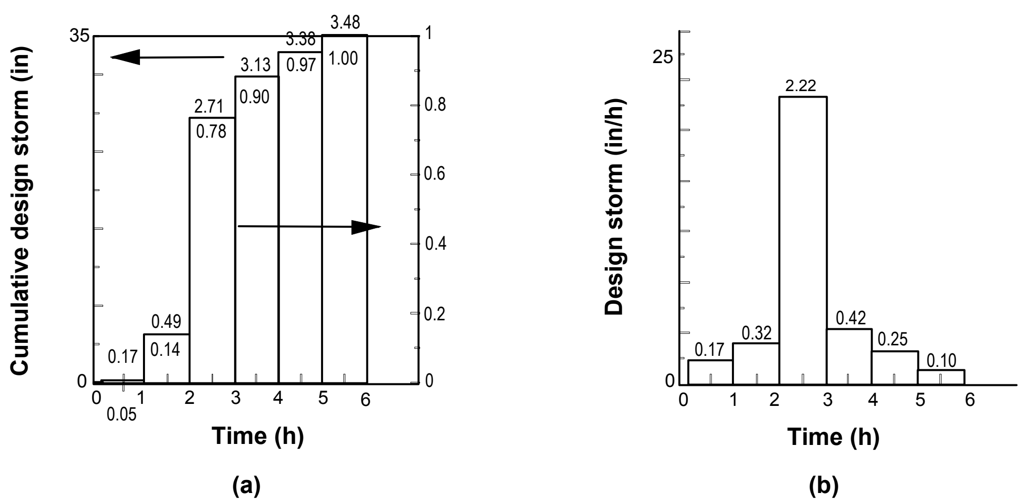
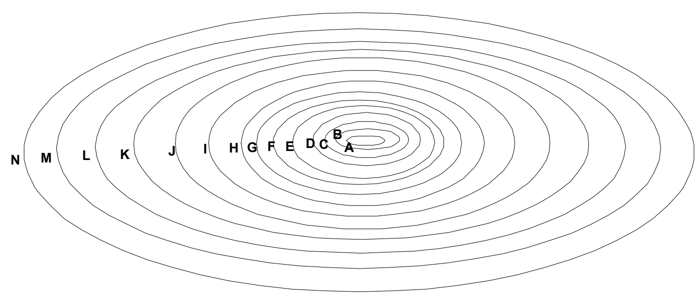


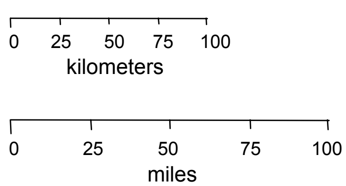
Figure 6.13(CU). 6-hour design storms for Example 6-6(CU): (a) dimensionless cumulative design storm and the cumulative design storm and (b) the design storm

6.2.3 Depth-Area Adjustments

The rainfall depths from IDF curves represent estimates for small areas. For designs on areas larger than a few square kilometers (miles), the point rainfall estimates obtained from IDF curves must be adjusted. The point estimates represent extreme values. As the spatial extent of a storm increases, the depth of rainfall decreases; storms have a spatial pattern as well as a temporal variation. Figure 6.14 shows a design storm rainfall pattern that is used in estimating probable maximum floods. Rainfall depths, which depend on the total rainfall and the depth-area relationship used at the location, decrease with increasing area. In Figure 6.14, the greatest depth would be for the innermost isohyet.

When selecting a point rainfall to apply uniformly over a watershed, the point value should be reduced to account for the areal extent of the storm. The reduction is made using a depth-area adjustment factor. The factor is a function of the drainage area and the rainfall duration. Figure 6.15 shows the depth-area adjustment factors based on Weather Bureau Technical Paper No. 40. This set of curves can be used unless specific curves derived from regional analyses are available. Figure 6.15 shows that the adjustment factor decreases from 100 percent as the watershed area increases and as the storm duration decreases. Beyond a drainage area of 800 km^2 (300 mi^2), the adjustment factor shows little change.





Isohyet Areas	1km²	(mi²)
A -	26	(10)
B -	65	(25)
C -	130	(50)
D -	259	(100)
E -	433	(167)
F -	777	(300)
G -	1166	(450)
H -	1813	(700)
I -	2590	(1,000)
J -	3885	(1,500)
K -	5568	(2,150)
L -	7770	(3,000)
M -	11655	(4,500)
N -	16835	(6,500)

Isohyet Areas Not Shown	1cm²	(mi²)
O -	25900	(10,000)
P -	38850	(15,000)
Q -	64750	(25,000)
R -	103600	(40,000)
S -	155400	(60,000)

Figure 6.14. Standard isohyetal pattern

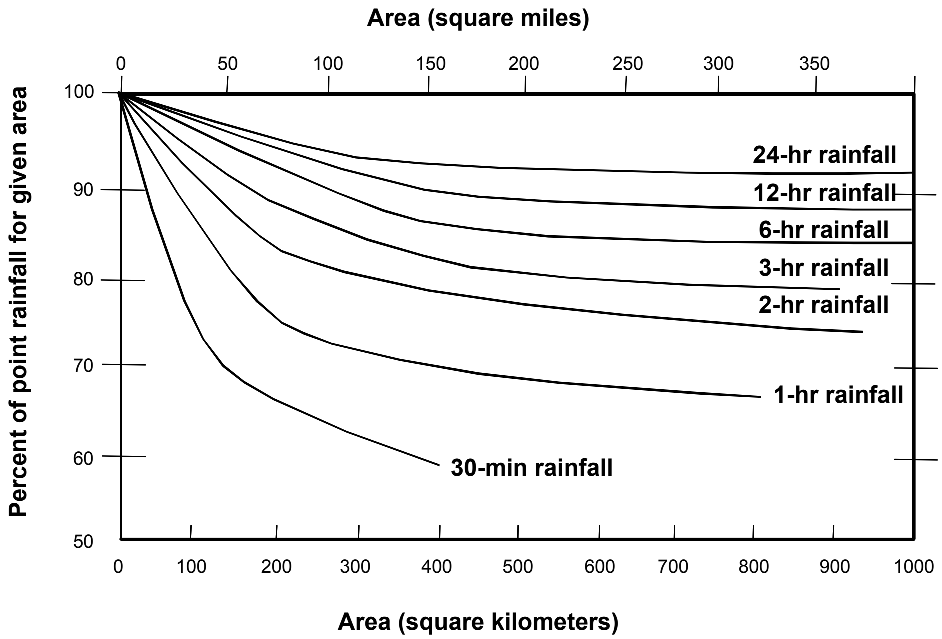


Figure 6.15. Depth-area curves for adjusting point rainfalls

6.2.4 Design Storm From Measured Storm Data

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 D 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.

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. Storms that produce floods of the desired frequency could be used to develop hyetographs. 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 that 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.

6.2.5 Design Storm by Triangular Hyetograph

In 1983, Yen and Chow developed a method for approximating a design storm hyetograph by a triangular distribution for watersheds smaller than 50 km^2 (20 mi^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 can be used as the average characteristics of an expected storm.

The triangular representation used by Yen and Chow (1983) is illustrated in Figure 6.16. 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 6.17 using the duration of the storm, t_d, and the total depth of rainfall, D, in mm (in). 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 critical duration of the design storm is assumed to equal the time of concentration so that the entire watershed would be contributing to the flow at the point of interest.

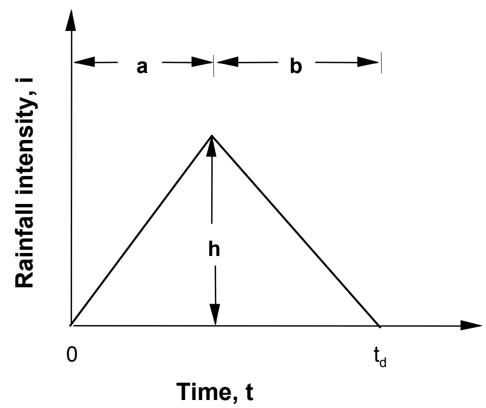


Figure 6.16. Triangular hyetograph

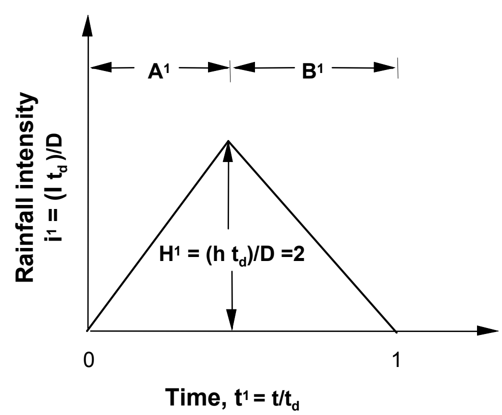


Figure 6.17. Normalized triangular hyetograph

Yen and Chow (1983) analyzed 293,946 storms from 222 National Weather Stations (NWS) and 13 Agricultural Research Service (ARS) rain gage stations to determine the statistical values of the normalized hyetograph parameters. They presented 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 rain gauge 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.

6.3 DESIGN HYDROGRAPH SYNTHESIS

For basins without measured data, synthetic methods can be used to develop unit hydrographs. These methods tend to be somewhat inflexible in that they use standard shapes for the unit hydrographs.

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 (Section 6.3.2) and the SCS method (Section 6.3.3).

As the name suggests, a design hydrograph is a hydrograph that has characteristics that are believed to represent the flood conditions at the limit considered acceptable. The design hydrograph is usually generated using a design-storm hyetograph and a unit hydrograph. However, the design hydrograph could also be an actual storm hydrograph that was experienced at the design location. In either case, the design hydrograph may have an exceedence frequency associated with it. In the case of design-storm modeling, it is common to assume that the frequency of the runoff hydrograph is the same as the frequency of the design hyetograph.

If precipitation and stream flow records are available for a particular design site, the development of the design hydrograph is a straightforward procedure. Unit hydrographs can be determined from the data using the methods described in Section 6.1. Rainfall records can be readily analyzed to determine the design hyetograph. Using the convolution process the unit hydrograph and the rainfall excess produce the direct-runoff hydrograph.

Example 6.7(SI). Sanders (1980) provides an example of developing a design hydrograph. In this example, the unit hydrograph ordinates have been determined for a 2-hour duration, and it is desired to compute the flood hydrograph for a complex storm over a 10-hour period. Figure 6.18(SI) shows the rainfall excess hyetograph (mm/h) and the unit hydrograph for the 16.2 km^2 watershed. The UH ordinates are given in column 2 of Table 6.7(SI). The intensities must be converted to depths prior to convolution. Since the intensities have units of mm/h and the time interval is 2 hours, the rainfall-excess depths are 7.6, 17.8, and 28.0 mm. The two periods in which there was no rainfall excess must be considered and the appropriate translation made when performing the convolution process. The base flow, which was initially separated out before determining the unit hydrograph, is added back to the direct runoff in order to determine the design hydrograph. This is shown in columns 8 and 9 of Table 6.7(SI).

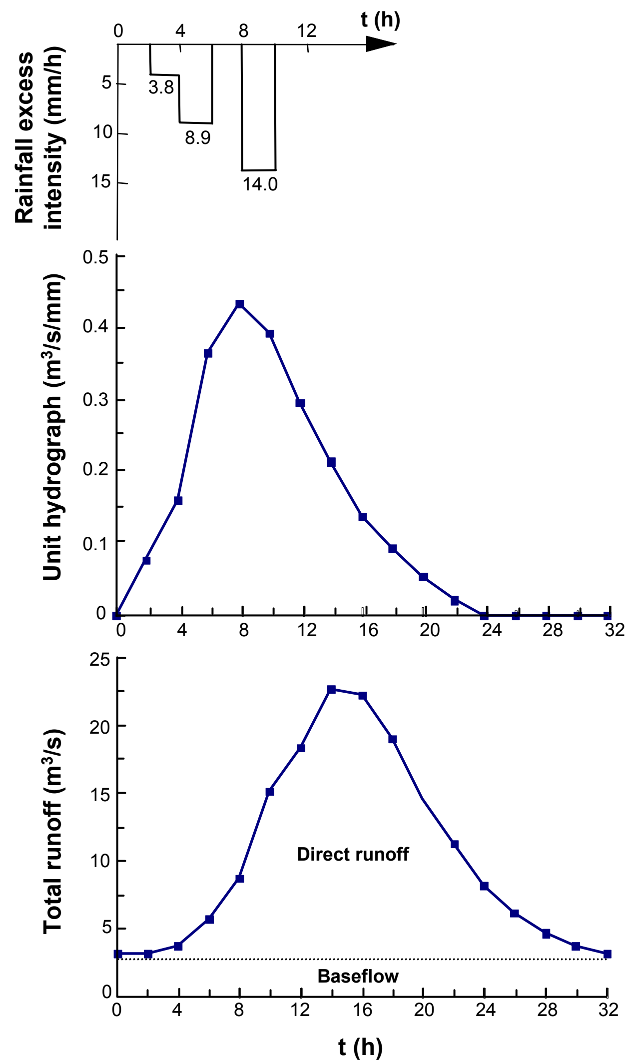


Figure 6.18(SI). Example of hydrograph synthesis

Table 6.7(SI). Calculation of Total Runoff by Convolving Rainfall Excess with the Unit Hydrograph

(1) Time (h)	(2) Unit Hydrograph (UH) ($m^3/s/mm$)	(3) UH 0.0	(4) UH 7.6	(5) UH 17.8	(6) UH 0.0	(7) UH 28.0	(8) Base Flow (m^3/s)	(9) Total Runoff (m^3/s)
0	0	0	0	0	0	0	3.12	3.12
2	0.077	0	0	0	0	0	3.12	3.12
4	0.160	0	0.59	0	0	0	3.12	3.71
6	0.366	0	1.22	1.37	0	0	3.12	5.71
8	0.434	0	2.78	2.85	0	0	3.12	8.75
10	0.393	0	3.30	6.51	0	2.16	3.12	15.09
12	0.297	0	2.99	7.73	0	4.48	3.12	18.32
14	0.214	0	2.26	7.00	0	10.25	3.12	22.63
16	0.137	0	1.63	5.29	0	12.15	3.12	22.19
18	0.094	0	1.04	3.81	0	11.00	3.12	18.97
20	0.055	0	0.71	2.44	0	8.32	3.12	14.59
22	0.022	0	0.42	1.67	0	5.99	3.12	11.20
24	0	0	0.17	0.98	0	3.84	3.12	8.11
26			0	0.39	0	2.63	3.12	6.14
28				0	0	1.54	3.12	4.66
30					0	0.62	3.12	3.74
32						0	3.12	3.12

Example 6.7(CU). Sanders (1980) provides an example of developing a design hydrograph. In this example the unit hydrograph ordinates have been determined for a 2-hour duration, and it is desired to compute the flood hydrograph for a complex storm over a 10-hour period. Figure 6.18(CU) shows the rainfall excess hyetograph (in/h) and the unit hydrograph for the 6.25 mi^2 watershed. The UH ordinates are given in column 2 of Table 6.7(CU). The intensities must be converted to depths prior to convolution. Since the intensities have units of in/h and the time interval is 2 hours, the rainfall-excess depths are 0.30, 0.70, and 1.1 in. The two periods in which there was no rainfall excess must be considered and the appropriate translation made when performing the convolution process. The base flow, which was initially separated out before determining the unit hydrograph, is added back to the direct runoff in order to determine the design hydrograph. This is shown in columns 8 and 9 of Table 6.7(CU).

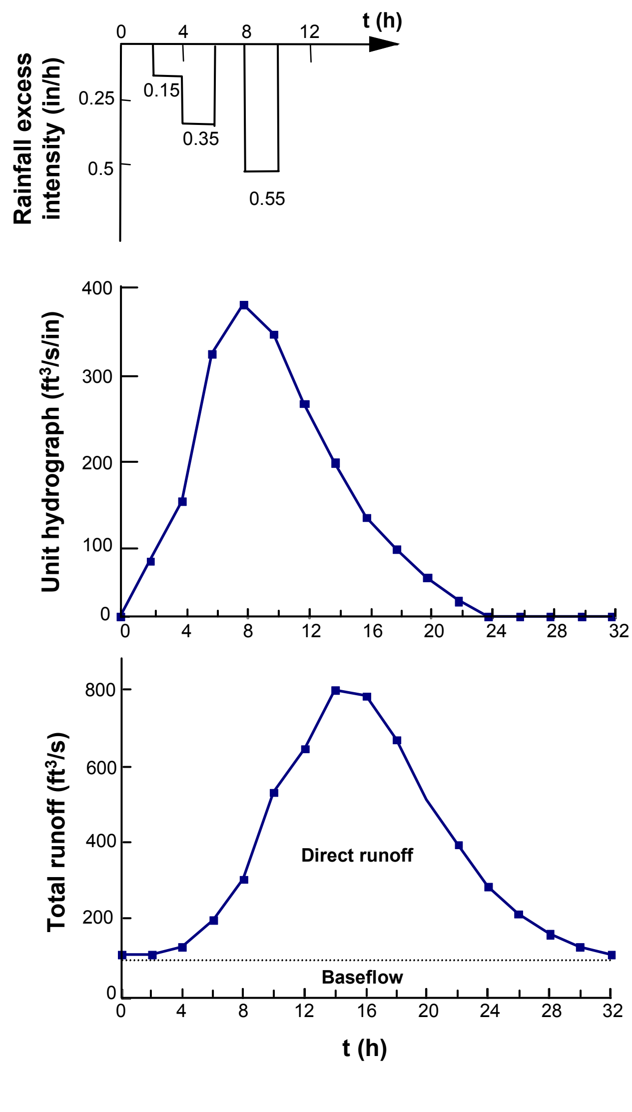


Figure 6.18(CU). Example of hydrograph synthesis

Table 6.7(CU). Calculation of Total Runoff by Convolving Rainfall Excess with the Unit Hydrograph

(1) Time (h)	(2) Unit hydrograph (UH) (ft³/s/in)	(3) UH 0.0	(4) UH 0.3	(5) UH 0.7	(6) UH 0.0	(7) UH 1.1	(8) Base Flow (ft³/s)	(9) Total Runoff (ft³/s)
	0	0					110	110
2	69	0	0				110	110
4	144	0	21	0			110	131
6	328	0	43	48	0		110	202
8	389	0	98	101	0	0	110	309
10	352	0	117	230	0	76	110	532
12	266	0	106	272	0	158	110	646
14	192	0	80	246	0	361	110	797
16	123	0	58	186	0	428	110	782
18	84	0	37	134	0	387	110	669
20	49	0	25	86	0	293	110	514
22	20	0	15	59	0	211	110	395
24	0	0	6	34	0	135	110	286
26			0	14	0	92	110	216
28				0	0	54	110	164
30					0	22	110	132
32						0	110	110

6.3.1 S-Hydrograph Method

Based on the unit hydrograph assumptions, it is possible to transform a unit hydrograph of specified duration into one with a different duration. The method of making the transform is called the S-hydrograph method; it is often referred to as the S-curve or S-graph method.

Suppose it is desired to find a 6-hour unit hydrograph from an existing 3-hour unit hydrograph (unit depth of excess rainfall in 3 hours). Unit depth equals 1 mm in SI units and 1 inch in CU units. Assuming independence of antecedent conditions, a second 3-hour unit graph is lagged or displaced 3 hours from the first as illustrated in Figure 6.19. The ordinates are then added which yields 2 times the unit depth of runoff in 6 hours. Dividing these ordinates by 2 gives the 6-hour unit hydrograph also shown in Figure 6.19. The division by 2 is necessary because the sum of the two 3-hour unit hydrographs produces a hydrograph with a depth twice the unit depth. It is important to recognize that the peak discharge of the 6-hour UH is lower than the peak of the 3-hour UH and that the time to peak is longer. This procedure is valid only when doubling the duration of a UH. In general, the S-hydrograph method should be used.

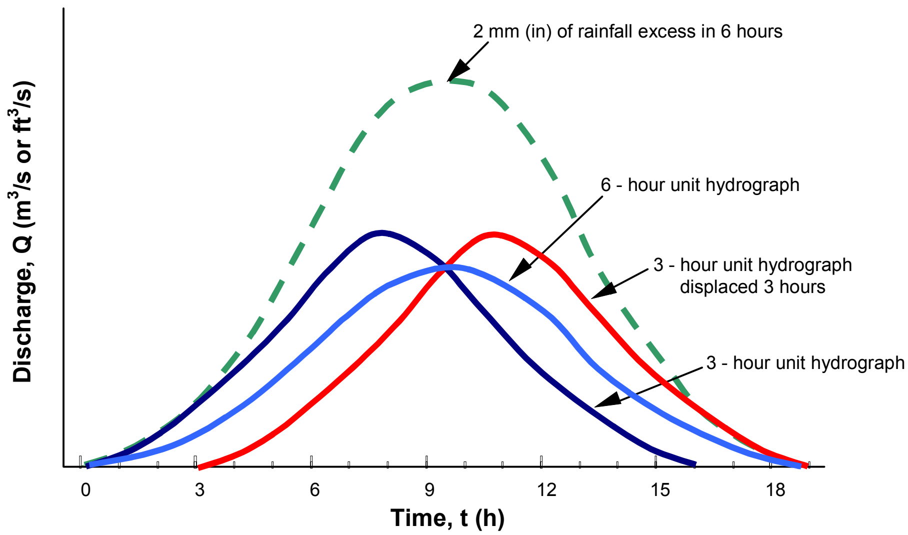


Figure 6.19. Development of a 6-hour unit hydrograph from a 3-hour unit hydrograph

To change the unit hydrograph from a longer duration to a shorter duration or to any duration that 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 6.20. The S-curve approaches a constant value of the discharge equal to (1 mm or 1 in) * (drainage area)/ 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 unit duration is obtained by lagging the S-curve by the new unit duration, subtracting the two S-curves from one another, and multiplying the resulting hydrograph ordinates by the ratio of the unit duration of the unit hydrograph used to construct the S-curve to the unit duration of the unit hydrograph being developed. For example, if a 3-hour unit hydrograph is to be developed from a 6-hour unit hydrograph, the ordinates are multiplied by two (2) to obtain a volume equal to 1 mm (1 in). Similarly, in going from 6 hours to 15 hours, the multiplier is 6/15 or 0.4.

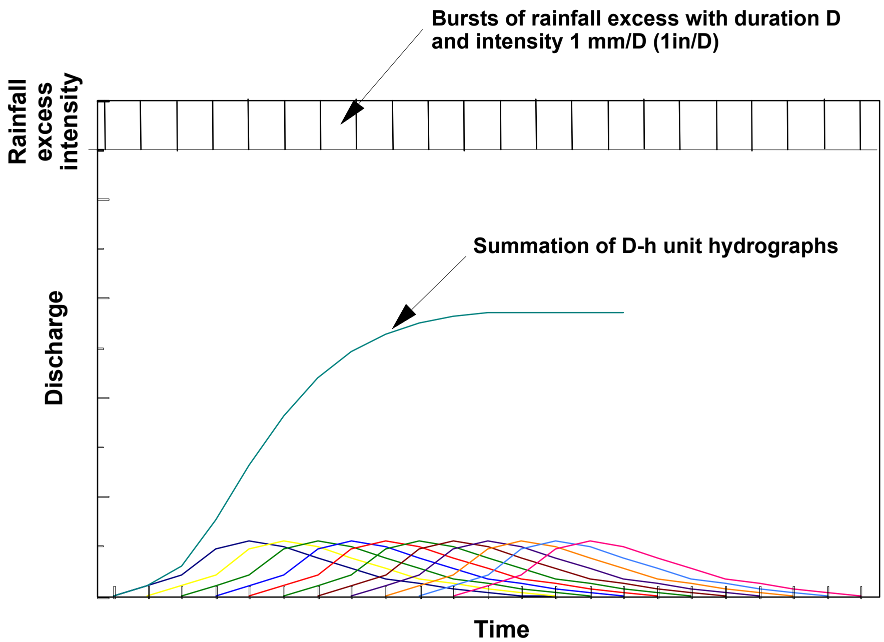


Figure 6.20. Graphical illustration of the S-curve construction

Example 6.8(SI). The ordinates of a 3-hour unit hydrograph are given in column 2 of Table 6.8(SI). The peak of 15.2 m^3/s occurs at the storm time of 12 hours. Assuming that a 6-hour UH is wanted, the S-curve is computed by forming the cumulative hydrograph (column 3). Since the desired 6-hour unit hydrograph has a unit duration of twice the unit duration of the 3-hour unit hydrograph, the S-curve is offset by two time intervals, or 6 hours (see column 4). (If a 9-hour unit hydrograph was needed, the offset would be three time intervals or 9 hours.) The difference between the S-curve (column 3) and the offset S-curve (column 4) is a 6-hour hydrograph (column 5) that has a volume of 2 mm. (If a 9-hour unit hydrograph was needed, the 9-hour hydrograph would have a volume of 3 mm.) A 6-hour unit hydrograph that has a volume of 1 mm (column 6) is computed by dividing the 6-hour hydrograph of column 5 by 2 mm.

Assuming that the drainage area of the watershed is 1242 km^2, the volume of the unit hydrograph can be computed with the trapezoidal rule:

$$\left(115\ \frac{m^3}{s}\right)(3\ h)\left(\frac{3600\ s}{h}\right)\left(\frac{1000\ mm}{m}\right)\left(\frac{1}{1242\ km^2}\right)\left(\frac{1\ km^2}{10^6\ m^2}\right) = 1\ mm$$

It is of interest to note that the peak of the 6-hour unit hydrograph is smaller than the peak of the 3-hour UH (i.e., 14.95 vs. 15.2 m^3/s/mm). The longer duration of rainfall excess is associated with greater smoothing or attenuation of the runoff, thus the smaller peak. The peak of the 6-hour UH also occurs 3 hours later than that of the 3-hour UH.

Table 6.8(SI). Computation of a 6-hour Unit Hydrograph from a 3-hour Unit Hydrograph Using the S-curve Method

(1) Time (h)	(2) 3-hour UH ($m^3/s/mm$)	(3) S-curve (m^3/s)	(4) Offset S-curve (m^3/s)	(5) 6-hour Hydrograph (m^3/s)	(6) 6-hour UH ($m^3/s/mm$)
0	0	0	0	0	0
3	3.7	3.7	0	3.7	1.85
6	9.6	13.3	0	13.3	6.65
9	13.1	26.4	3.7	22.7	11.35
12	15.2	41.6	13.3	28.3	14.15
15	14.7	56.3	26.4	29.9	14.95
18	13.3	69.6	41.6	28.0	14.00
21	11.8	81.4	56.3	25.1	12.55
24	9.4	90.8	69.6	21.2	10.60
27	7.9	98.7	81.4	17.3	8.65
30	6.0	104.7	90.8	13.9	6.95
33	4.5	109.2	98.7	10.5	5.25
36	3.1	112.3	104.7	7.6	3.80
39	1.9	114.2	109.2	5.0	2.50
42	0.8	115.0	112.3	2.7	1.35
45	0.0	115.0	114.2	0.8	0.40
48	0.0	115.0	115.0	0.0	0.00
					Sum = 115.00

Example 6.8(CU). The ordinates of a 3-hour unit hydrograph are given in column 2 of Table 6.8(CU). The peak of 13,600 ft^3/s occurs at the storm time of 12 hours. Assuming that a 6-hour UH is desired, the S-curve is computed by forming the cumulative hydrograph (column 3). Since the desired 6-hour unit hydrograph has a unit duration of twice the unit duration of the 3-hour unit hydrograph, the S-curve is offset by two time intervals, or six hours (see column 4). (If a 9-hour unit hydrograph was needed, the offset would be three time intervals or 9 hours.) The difference between the S-curve (column 3) and the offset S-curve (column 4) is a 6-hour hydrograph (column 5) that has a volume of 2 in. (If a 9-hour unit hydrograph was needed, the 9-hour hydrograph would have a volume of 3 in.) A 6-hour unit hydrograph that has a volume of 1 in (column 6) is computed by dividing the 6-hour hydrograph of column 5 by 2 in.

Assuming that the drainage area of the watershed is 479.5 mi^2, the volume of the unit hydrograph can be computed with the trapezoidal rule:

$$\left(103{,}000\ \frac{ft^3}{s}\right)(3\ h)\left(\frac{3600\ s}{h}\right)\left(\frac{12\ in}{ft}\right)\left(\frac{1}{479.5\ mi^2}\right)\left(\frac{1\ mi^2}{27{,}878{,}400\ ft^2}\right) = 1\ in$$

It is of interest to note that the peak of the 6-hour unit hydrograph is smaller than the peak of the 3-hour UH (i.e., 13,400 vs. 13,600 ft^3/s/in). The longer duration of rainfall excess is associated with greater smoothing or attenuation of the runoff, thus the smaller peak. The peak of the 6-hour UH also occurs 3 hours later than that of the 3-hour UH.

Table 6.8(CU). Computation of a 6-hour Unit Hydrograph from a 3-hour Unit Hydrograph Using the S-curve Method

(1) Time (h)	(2) 3-hour UH (ft^3/s/in)	(3) S-Curve (ft^3/s)	(4) Offset S-curve (ft^3/s)	(5) 6-hour Hydrograph (ft^3/s)	(6) 6-hour UH (ft^3/s/in)
0	0	0		0	0
3	3,300	3,300		3,300	1,650
6	8,600	11,900	0	11,900	5,950
9	11,700	23,600	3,300	20,300	10,150
12	13,600	37,200	11,900	25,300	12,650
15	13,200	50,400	23,600	26,800	13,400
18	11,900	62,300	37,200	25,100	12,550
21	10,600	72,900	50,400	22,500	11,250
24	8,400	81,300	62,300	19,000	9,500
27	7,100	88,400	72,900	15,500	7,750
30	5,400	93,800	81,300	12,500	6,250
33	4,000	97,800	88,400	9,400	4,700
36	2,800	100,600	93,800	6,800	3,400
39	1,700	102,300	97,800	4,500	2,250
42	700	103,000	100,600	2,400	1,200
45	0	103,000	102,300	700	350
48	0	103,000	103,000	0	0
					Sum = 103,000

6.3.2 Snyder Unit Hydrograph

This method developed in 1938 has been used extensively by the Corps of Engineers. 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 D-hour unit hydrograph. The entire time distribution of the unit hydrograph is not explicitly determined using this method, but seven points are given through which a smooth curve can be drawn.

Certain key parameters of the unit hydrograph are evaluated and from these a characteristic unit hydrograph is constructed. The key parameters 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 mm (1 in) of runoff. If it does not, the ordinates are adjusted accordingly. A typical Snyder hydrograph is shown in Figure 6.21.

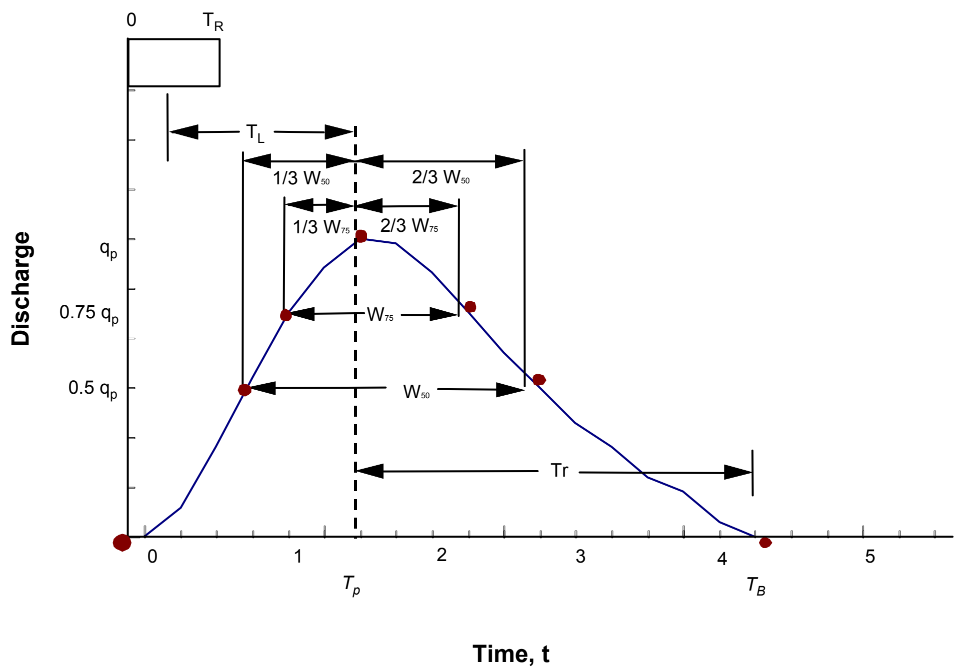


Figure 6.21. Snyder synthetic unit hydrograph definition

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 from other studies and textbooks or by analyzing unit hydrographs derived for gauged streams in the same general area. Another source of information is the Corps of Engineers, District Offices. C_t is a coefficient that represents the variation of unit hydrograph lag time with watershed slope 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 that represents the variation of the 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, the length along the main channel from the outlet to the divide, L, and the length along the main channel to a point opposite the watershed centroid, L_{ca} 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. The following empirical equation is used to estimate the lag time:

$$T_L = \alpha C_t \,(L\, L_{ca})^{0.3} \qquad (6.8)$$

   where,
   T_L = lag time, h
   C_t = empirical coefficient
   L = length along main channel from outlet to divide, km (mi)
   L_{ca} = length along main channel from outlet to a point opposite the watershed centroid, km (mi)
   α = conversion constant (0.75 for SI units and 1.00 for CU units).

3. Determine unit duration of the unit hydrograph:

   The relationship developed by Snyder for the unit duration of the excess rainfall, T_R in hours, is a function of the lag time computed above, namely:

$$T_R = \frac{T_L}{5.5} \qquad (6.9)$$

   Equation 6.9 provides an initial value of T_R.

   A relationship has been developed to adjust the computed lag time for other unit durations. This is necessary because the equation above may result in inconvenient values of the unit duration. The adjustment relationship is:

$$T_{L(adj.)} = T_L + 0.25\left(T_R' - T_R\right) \qquad (6.10)$$

   where,
   $T_{L(adj.)}$ = adjusted lag time for the new duration, h
   T_L = original lag time as computed above, h
   T_R = original unit duration (i.e., Equation 6.9), h

T_R' = desired unit duration, h.

As an example, if the originally computed lag time, T_L, was 12.5 hours, then the corresponding unit duration would be (12.5/5.5) or 2.3 hours. It would be more convenient to have a unit duration of 2.0 hours, so the lag time is adjusted as follows:

$$\begin{aligned} T_{L(adj)} &= T_L + 0.25\,(T_R' - T_R) \\ &= 12.5 + 0.25(2.0 - 2.3) \\ &= 12.4\ h \end{aligned}$$

An alternative procedure would be to use the S-curve method (Section 6.3.1) to convert the 2.3-hour UH to a 2.0-hour UH, but the above empirical procedure is much simpler.

4. Determine peak discharge:

The peak discharge for the unit hydrograph is determined from the following equation:

$$q_P = \alpha \frac{C_P A}{T_{L(adj)}} \tag{6.11}$$

where,

q_p = unit peak discharge, m³/s/mm (ft³/s/in)
C_p = empirical coefficient
A = watershed area, km^2 (mi^2)
α = conversion constant (0.275 for SI and 640 for CU units).

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:

$$T_B = 3 + \frac{T_{L(adj)}}{8} \tag{6.12}$$

where,

T_B = time of the synthetic unit hydrograph, 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, are approximated by the following equations

$$W_{50} = \alpha_{50} \left(\frac{q_p}{A} \right)^{-1.075} \quad (6.13)$$

and

$$W_{75} = \alpha_{75} \left(\frac{q_p}{A} \right)^{-1.075} \quad (6.14)$$

where,

W_{50} = time interval between the rising and falling limbs at 50% of peak discharge, h
W_{75} = time interval between the rising and falling limbs at 75% of peak discharge, h
q_p = unit peak discharge, $m^3/s/mm$ ($ft^3/s/in$)
A = watershed area, km^2 (mi^2)
α_{25} = unit conversion constant (0.18 in SI and 735 in CU units)
α_{75} = unit conversion constant (0.10 in SI and 434 in CU units).

7. Construct unit hydrograph:

   Using the values computed in the previous steps, the unit hydrograph can now be sketched, remembering that the total depth of runoff must equal 1mm (1 in). 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.

Example 6.9. A synthetic unit hydrograph is to be constructed for a watershed of 2,266 km^2,(875 mi^2) where L is measured to be 133.6 km (83 mi) and L_{ca} is 65 km (40 mi). For this region, average values of C_t = 1.32 and C_p = 0.63 have been found to apply. Therefore, T_L and T_R are:

Variable	Value in SI	Value in CU
$T_L = \alpha 75 \ C_t \ (L L_{ca})^{0.3}$	$= 0.75 \times 1.32(133.6 \times 65)^{0.3}$ $= 15.0 \ h$	$= 1.0 \times 1.32(83 \times 40)^{0.3}$ $= 15.0 \ h$

$$T_R = \frac{T_L}{5.5} = 2.7 \ h \ (use \ 3 \ h)$$

A 3-hour unit hydrograph is desired:

$$T_{L(adj)} = T_L + 0.25 \ (T_{R'} - T_R) = 15.0 + 0.25(3 - 2.7) = 15.1h$$

$$T_B = 3 + \frac{T_{L(adj)}}{8} = 3 + \frac{15.1}{8} = 4.9 \ days = 118h$$

Variable	Value in SI	Value in CU
$q_P = a \frac{C_P A}{T_{L(adj)}}$	$= \frac{(0.275)(0.63)(2,266)}{15.1} = 26 m^3/s/mm$	$= \frac{(640)(0.63)(875)}{15.1} = 23,000 ft^3/s/in$
$W_{50} = \alpha \left(\frac{q_p}{A}\right)^{-1.075}$	$= 0.18\left(\frac{26}{2266}\right)^{-1.075} = 22\ h$	$= 735\left(\frac{23,000}{875}\right)^{-1.075} = 22\ h$
$W_{75} = \alpha \left(\frac{q_p}{A}\right)^{-1.075}$	$= 0.10\left(\frac{26}{2266}\right)^{-1.075} = 12.2\ h$ (use 12 h)	$= 434\left(\frac{23,000}{875}\right)^{-1.075} = 13\ h$

Compared to the hydrograph widths at 50 and 75 percent of the peak flow, a time base of 118 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

$$T_B = 4.5(T_{L(adj)} + \frac{1}{2}T_R) = 4.5\left[15.1 + \frac{1}{2}(2.7)\right] = 74.0\ h\ (use\ 74\ h)$$

These points are plotted in Figure 6.22, and a smooth hydrograph shape is fitted with the key dimensions. The volume under the unit hydrograph is then computed using the trapezoidal rule:

In SI:

$$\Delta t\sum_{i=1}^{25} q_{P_i} = \left(178.8\,\frac{m^3}{s/mm}\right)(3\,h)\left(\frac{3600\,s}{h}\right)\left(\frac{1}{2266\,km^2}\right)\left(\frac{1\,km}{1000\,m}\right)^2\left(\frac{1000\,mm}{m}\right)(1\,mm) = 0.85\,mm$$

In CU:

$$\Delta t\sum_{i=1}^{25} q_{P_i} = \left(160,300\,\frac{ft^3}{s/in}\right)(3\,h)\left(\frac{3600\,s}{h}\right)\left(\frac{1}{875\,mi^2}\right)\left(\frac{1\,mi}{5280\,ft}\right)^2\left(\frac{12\,in}{ft}\right)(1\,in) = 0.85\,mm$$

with the discharge ordinates being scaled from the figure and listed in column 2 of Table 6.9. The total volume is 0.85 units, which is less than the required unit depth. The volume must be increased to the unit depth in a reasonable and systematic way. The procedure described below is recommended for the following reasons: (1) the times to peak and peak discharge are preserved; (2) the bulk of the volume is added to 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 Equation 6.12.

Since the first estimate of the unit hydrograph was approximately 15 percent less than the necessary unit depth, a second approximation was sketched (see Figure 6.22) with the ordinates given in column 3 of Table 6.9. The depth was computed using the trapezoidal rule:

In SI:

$$\Delta t\sum_{i=1}^{25} q_{P_i} = \left(202.8\,\frac{m^3}{s/mm}\right)(3\,h)\left(\frac{3600\,s}{h}\right)\left(\frac{1}{2266\,km^2}\right)\left(\frac{1\,km}{1000\,m}\right)^2\left(\frac{1000\,mm}{m}\right)(1\,mm) = 0.97\,mm$$

In CU:

$$\Delta t\sum_{i=1}^{25} q_{P_i} = \left(181,800\,\frac{ft^3}{s / in}\right)(3\,h)\left(\frac{3600\,s}{h}\right)\left(\frac{1}{875\,mi^2}\right)\left(\frac{1\,mi}{5280\,ft}\right)^2\left(\frac{12\,in}{ft}\right)(1\,in) = 0.97\,mm$$

Thus, the second approximation is about 3 percent less than the required unit depth. The ordinates on the rising and recession limbs were modified by systematically adding to the ordinates, with the resulting unit hydrograph given in column 4 of Table 6.9. The resulting unit hydrograph has a unit depth.

The final unit hydrograph (see Figure 6.22 and Table 6.9) is a 3-hour unit. It can be used in the same manner as a unit hydrograph derived from gage records.

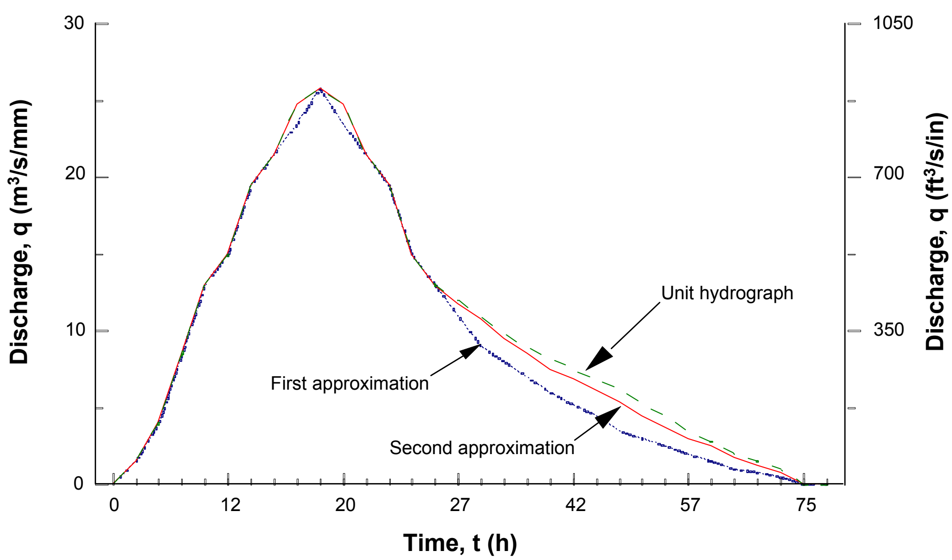


Figure 6.22. Unit hydrograph analysis

Table 6.9. Adjustment of Ordinates of Snyder's Unit Hydrograph

	SI			CU		
(1) **Time (h)**	**(2)** **Initial Hydrograph ($m^3/s/mm$)**	**(3)** **Second Hydrograph ($m^3/s/mm$)**	**(4)** **Third Hydrograph ($m^3/s/mm$)**	**(2)** **Initial Hydrograph ($ft^3/s/in$)**	**(3)** **Second Hydrograph ($ft^3/s/in$)**	**(4)** **Third Hydrograph ($ft^3/s/in$)**
0	0	0	0	0	0	0
3	1.7	2.3	2.4	1,500	2,100	2,200
6	4.0	5.2	5.4	3,600	4,700	4,800
9	8.5	9.9	10.1	7,600	8,900	9,100
12	15.3	15.3	15.4	13,700	13,700	13,800
15	21.5	23.0	23.0	19,300	20,600	20,600
18	25.9	25.9	25.9	23,000	23,000	23,000
21	21.5	22.4	22.4	19,300	20,100	20,100
24	15.0	15.0	15.0	13,500	13,500	13,500
27	11.0	11.8	11.9	9,900	10,600	10,700
30	9.2	10.6	10.8	8,300	9,500	9,700
33	7.9	9.4	9.7	7,100	8,400	8,700
36	6.8	8.5	8.9	6,100	7,600	8,000
39	6.0	7.6	8.1	5,400	6,800	7,300
42	5.2	6.8	7.4	4,700	6,100	6,600
45	4.4	6.1	6.8	3,900	5,500	6,100
48	3.7	5.3	6.1	3,300	4,800	5,500
51	3.0	4.5	5.2	2,700	4,000	4,700
54	2.4	3.8	4.4	2,200	3,400	3,900
57	1.9	3.0	3.5	1,700	2,700	3,100
60	1.4	2.4	2.8	1,300	2,200	2,500
63	1.0	1.7	2.0	900	1,500	1,800
66	0.7	1.1	1.3	600	1,000	1,200
69	0.5	0.8	0.9	400	700	800
72	0.3	0.4	0.4	300	400	400
75	0	0	0	0	0	0
	178.8	201.72	209.8	160,300	181,800	188,100

6.3.3 SCS Unit Hydrograph

The Soil Conservation Service Handbook (1972) presents a synthetic unit hydrograph procedure that 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 input parameters are the peak discharge, the area of the watershed, and the time to peak. With these parameters, a standard unit hydrograph is constructed.

The SCS methods use dimensionless unit hydrographs that are based on an extensive analysis of measured data. Unit hydrographs were evaluated for a large number of actual watersheds and then made dimensionless by dividing all discharge ordinates by the peak discharge and the time ordinates by the time to peak. An average of these dimensionless unit hydrographs (UH) was computed. The time base of the dimensionless UH was approximately 5 times the time to peak, and approximately 3/8 of the total volume occurred before the time to peak; the inflection point on the recession limb occurs at approximately 1.7 times the time to peak, and the UH has a curvilinear shape. The dimensionless UH is shown in Figure 6.23 and the discharge ratios for selected values of the time ratios are given in Table 6.10.

For purposes of comparison, the curvilinear unit hydrograph can be approximated by a triangular UH that has similar characteristics; Figure 6.24 shows a comparison of the two dimensionless unit hydrographs. It is important to recognize that the triangular UH is not a substitute for the curvilinear UH. The curvilinear UH is always used in hydrologic computations. The triangular unit hydrograph is only used to develop an expression for computing the peak

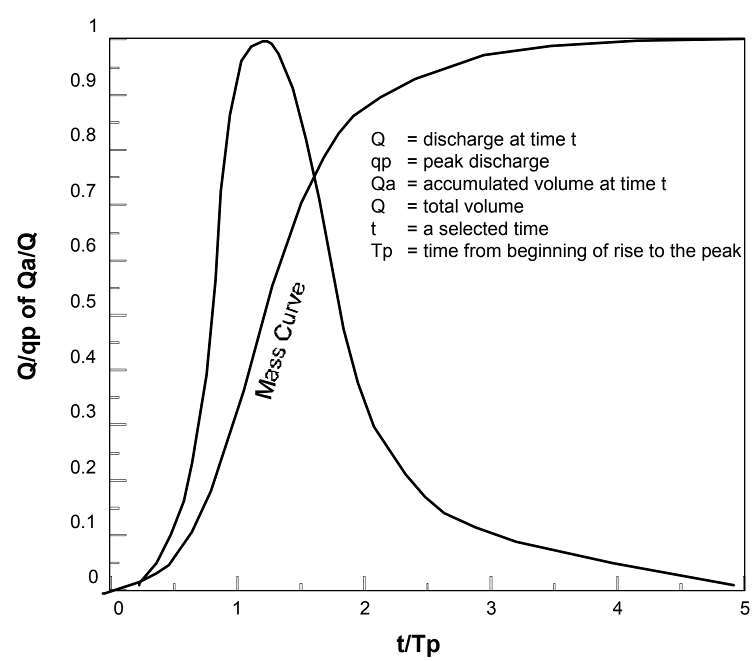


Figure 6.23. Dimensionless unit hydrograph and mass curve

discharge of the curvilinear unit hydrograph. While the time base of the triangular UH is only 8/3 of the time to peak (compared to 5 for the curvilinear UH), the areas under the rising limbs of the two UHs are the same (i.e., 37.5 percent).

Table 6.10. Ratios for Dimensionless Unit Hydrograph and Mass Curve

Time Ratios t/T_p	Discharge Ratios q/q_p	Mass Curve Ratios Q_a/Q
0	0.000	0.000
0.1	0.030	0.001
0.2	0.100	0.006
0.3	0.190	0.012
0.4	0.310	0.035
0.5	0.470	0.065
0.6	0.660	0.107
0.7	0.820	0.163
0.8	0.930	0.228
0.9	0.990	0.300
1.0	1.000	0.375
1.1	0.990	0.450
1.2	0.930	0.522
1.3	0.860	0.589
1.4	0.780	0.650
1.5	0.680	0.700
1.6	0.560	0.751
1.7	0.460	0.790
1.8	0.390	0.822
1.9	0.330	0.849
2.0	0.280	0.871
2.2	0.207	0.908
2.4	0.147	0.934
2.6	0.107	0.953
2.8	0.077	0.967
3.0	0.055	0.977
3.2	0.040	0.984
3.4	0.029	0.989
3.6	0.021	0.993
3.8	0.015	0.995
4.0	0.011	0.997
4.5	0.005	0.999
5.0	0.000	1.000

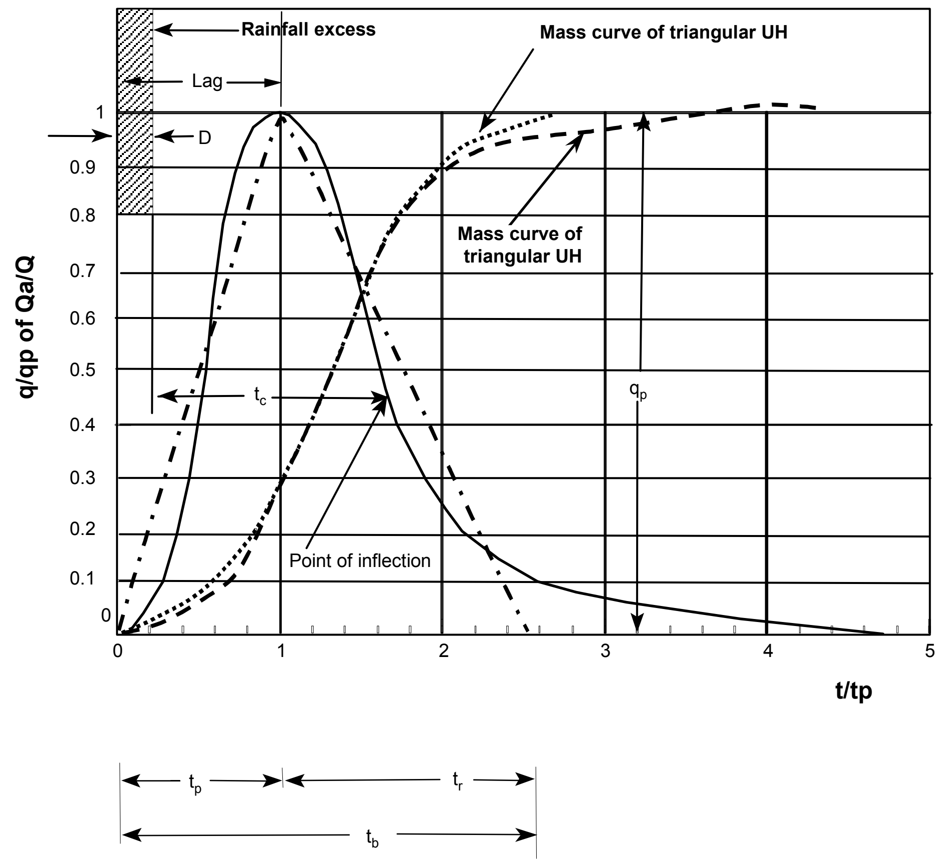


Figure 6.24. Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph

The area under a hydrograph equals the depth of direct runoff Q, which is 1 mm (1 in) for a unit hydrograph; based on geometry the runoff volume is related to the characteristics of the triangular unit hydrograph by

$$AQ = \frac{1}{2} q_p (t_p + t_r) \tag{6.15}$$

where,

t_p = time to peak
t_r = recession time
A = watershed area
q_p =peak discharge.

Solving Equation 6.15 for q_p and rearranging yields:

$$q_p = \frac{AQ}{t_p}\left[\frac{2}{1 + t_r / t_p}\right] \tag{6.16}$$

Letting K_p replace the contents within the brackets yields:

$$q_p = \frac{K_p AQ}{t_p} \tag{6.17}$$

Equation 6.17 was developed using CU units of ft^3/s for discharge, mi^2 for area, in for runoff depth, and hours for t_p and setting $t_r = 1.67t_p$. Therefore, K_p equals 484. A more general expression is:

$$q_p = \frac{\alpha K_p AQ}{t_p} \tag{6.18}$$

where,

q_p = peak discharge, m^3/s (ft^3/s)
A = watershed area, km^2 (mi^2)
Q = runoff depth, mm (in)
t_p = time to peak, h
K_p = peaking constant equal to 484, dimensionless
α = unit conversion constant equal to 0.00043 in SI units and 1 in CU units.

By definition, the constant K_p equal to 484 reflects a hydrograph that has 3/8 of its area under the rising limb. Equation 6.18 may be applied to determine the peak of the curvilinear unit hydrograph only when K_p = 484 and Q = 1 (mm or in). In mountainous or flat areas, it is reasonable to expect that the volume fraction under the rising limb will change. For mountainous watersheds, the volume fraction could be expected to be greater than 3/8, and therefore the peaking constant may be near 600 while for flat, wetland areas the constant may be on the order of 300. However, if other peaking factors are used, *the curvilinear dimensionless unit hydrograph cannot be used because the volume under the hydrograph will not be equal to 1.* A triangular approximation of the unit hydrograph may be applied that adjusts the receding limb so that the volume equals 1.

The time to peak of Equation 6.18 can be expressed in terms of the unit duration of the rainfall excess and the time of concentration. Figure 6.24 provides the following two relationships:

$$t_c + D = 1.7\, t_p \tag{6.19}$$

and if the lag equals 0.6 t_c, then

$$\frac{D}{2} + 0.6\, t_c = t_p \tag{6.20}$$

Solving Equations 6.19 and 6.20 for D yields:

$$D = \frac{2}{15}\, t_c = 0.133\, t_c \tag{6.21}$$

Therefore, t_p can be expressed in terms of t_c:

$$t_p = \frac{D}{2} + 0.6\, t_c = \frac{2}{3} t_c \cong 0.667\, t_c \tag{6.22}$$

Expressing Equation 6.18 in terms of t_c rather than t_p yields

$$q_p = \frac{\alpha K_P\, AQ}{0.667\ t_c} \tag{6.23}$$

Example 6.10. The objective is to determine the curvilinear UH for a 1.2 km^2 (0.46 mi^2) watershed that has been commercially developed. The flow length is 1982 m (6500 ft), the slope is 1.3 percent, and the soil is of group B. Assume that a time of concentration of 1.34 hours was computed.

For commercial land use and soil group B, the watershed CN is 92 (see Table 5.4). For a unit of rainfall excess (1 mm in SI and 1 inch in CU units), Equation 6.23 provides a peak discharge:

Variable	Value in SI	Value in Cu
$q_p = \frac{\alpha K_P\ AQ}{0.667\ t_c}$	$= \frac{0.00043\,(484)(1.2\ km^2)(1\ mm)}{(0.667)(1.34\ h)}$ $= 0.28\ m^3 / s / mm$	$= \frac{1.0\,(484)(0.46\ mi^2)(1\ in)}{(0.667)(1.34\ h)}$ $= 250\ ft^3 /s/in$

The time to peak is:

$$t_p = \frac{2}{3}\ t_c = 0.89\ h$$

and the time base of the UH is:

$$t_b = 5\, t_p = 4.5\ h$$

The ordinates of the SCS curvilinear UH can be determined using the values of Table 6.10. The curvilinear UH will be approximated for selected values of t/t_p; the SCS TR-20 (1984) computer program uses all of the values shown in Table 6.10. For selected values of t/t_p, the curvilinear UH is computed in Table 6.11 and shown in Figure 6.25. The curvilinear UH can be considered a D-hour UH, with D computed by Equation 6.21 as:

$$D = 0.133\, t_c = 0.18\ h$$

Thus the UH should be reported on an interval of 0.18 hour, or about 0.2 hour, and all computations performed at that interval.

Table 6.11. Calculation of SCS Curvilinear Unit Hydrograph

t/t_p	q/q_p	t (h)	q (m^3/s/mm)	q (ft^3/s/in)
0.0	0.000	0	0	0
0.4	0.310	0.357	0.087	78
0.7	0.820	0.625	0.230	205
1.0	1.000	0.893	0.280	250
1.5	0.680	1.340	0.190	170
2.0	0.280	1.786	0.078	70
3.0	0.055	2.679	0.015	14
4.0	0.011	3.572	0.003	3
5.0	0.000	4.465	0	0

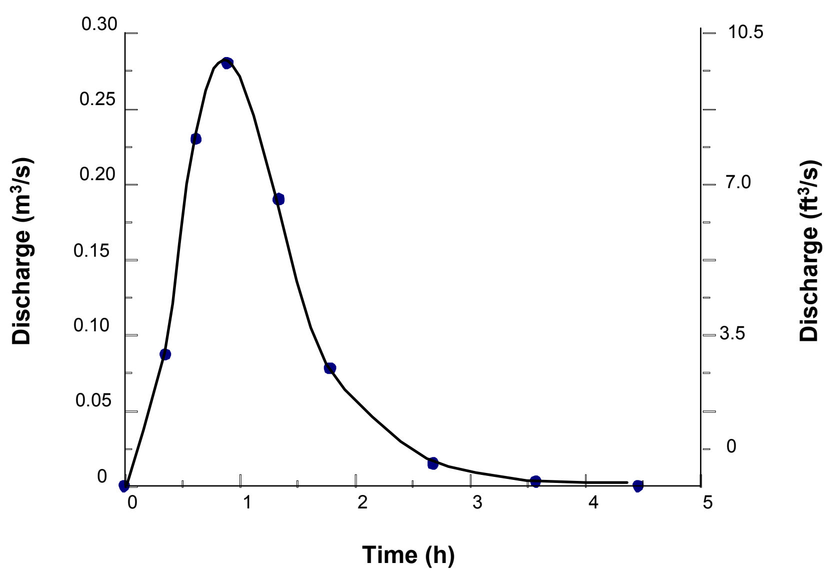


Figure 6.25. Example: SCS curvilinear unit hydrograph

6.3.4 Rainfall Excess Determination: SCS Method

In Section 6.1.4.3, the phi-index method was introduced for estimating loss functions. In hydrograph synthesis where the hyetograph is a design storm, it is necessary to extract losses to produce the rainfall-excess hyetograph. When developing a design hydrograph for a watershed on which unit hydrograph analyses have been made, it might be appropriate to use the mean of the phi-index values computed for the storm events used in the analysis phase. Some attempts have been made to relate mean values of the phi-index to soil characteristics but there are no generally accepted values.

SCS uses their rainfall-runoff equation (Equation 5.17) to compute a loss function that is applied to their design storms. Specifically, the procedure is as follows:

1. Determine the weighted curve number for the watershed.

2. Determine the 24-hour rainfall depth (mm or in) at the design exceedence frequency from the local intensity-duration-frequency curve, where depth equals the product of intensity and duration.

3. Multiply the 24-hour rainfall depth and each ordinate of the appropriate cumulative SCS design storm, which produces a cumulative 24-hour design storm.

4. For each ordinate of the design storm, use the cumulative precipitation as P in Equation 5.17 to compute the depth of rainfall excess Q. The difference between the cumulative P and cumulative Q is the loss function.

In this case, the rainfall excess is computed directly from the total rainfall hyetograph rather than computing a loss function and then subtracting it from the hyetograph to get the distribution of rainfall excess.

Example 6.11. In practice, the computations are made for a 24-hour duration on a time increment as small as 1 minute. While this is easily done by a computer program, a much simpler example will be used to illustrate the steps of the procedure given above.

To illustrate the process, a 12-hour dimensionless design storm with the following 2-hour ordinates is assumed: 0.087, 0.239, 0.543, 0.804, 0.935, 1.000. The design CN is 80, and the design rainfall depth is 92 mm (3.6 in). For a CN of 80, Equation 5.18 gives S = 63.5 mm (2.5 in) and Equation 5.16 gives an initial abstraction of 12.7 mm (0.5 in). Thus, the initial abstraction consists of all of the rainfall in the first 2 hours (i.e., 8.0 mm (0.31 in), and 4.7 mm (0.19 in) in the second 2-hour period; see columns 5 and 6 of Table 6.12). For those increments where the cumulative rainfall exceeds I_a, the cumulative rainfall excess is computed using Equation 5.17. For example, at a storm time of 6 hours, the cumulative rainfall is 50 mm (1.95 in) (column 3 of Table 6.12). Thus, the cumulative rainfall excess is:

	Value in SI	Value in CU
$Q = \dfrac{(P - I_a)^2}{P + S - I_a}$	$= \dfrac{(50 - 12.7)^2}{50 + 63.5 - 12.7} = 13.8\ mm$	$= \dfrac{(1.95 - 0.5)^2}{1.95 + 2.5 - 0.5} = 0.53\ in$

Table 6.12. Computation of Rainfall-Excess Hyetograph Using SCS Rainfall-Runoff Equation

(a) SI Units

(1) Time (h)	(2) Cumulative Dimensionless Hyetograph	(3) Cumulative Design Hyetograph (mm)	(4) Design Hyetograph (mm)	(5) Cumulative Initial Abstraction (mm)	(6) Incremental Initial Abstraction (mm)	(7) Cumulative Rainfall Excess (mm)	(8) Rainfall Excess Hyetograph (mm)	(9) Other Losses (mm)	(10) Cumulative Other Losses (mm)	(11) Total Loss Function (%)
2	0.087	8	8	8.0	8.0	0	0	0	0	100
4	0.239	22	14	12.7	4.7	1.2	1.2	8.1	8.1	91
6	0.543	50	28	-	-	13.8	12.6	15.4	23.5	55
8	0.804	74	24	-	-	30.1	16.3	7.7	31.2	32
10	0.935	86	12	-	-	39.3	9.2	2.8	34.0	23
12	1.000	92	6	-	-	44.0	4.7	1.3	35.3	22
Sum			92		12.7		44.0	35.3		

(b) CU Units

(1) Time (h)	(2) Cumulative Dimensionless Hyetograph	(3) Cumulative Design Hyetograph (in)	(4) Design Hyetograph (in)	(5) Cumulative Initial Abstraction (in)	(6) Incremental Initial Abstraction (in)	(7) Cumulative Rainfall Excess (in)	(8) Rainfall Excess Hyetograph (in)	(9) Other Losses (in)	(10) Cumulative Other Losses (in)	(11) Total Loss Function (%)
2	0.087	0.31	0.31	0.31	0.31	0.00	0.00	0.00	0.00	100
4	0.239	0.86	0.55	0.50	0.19	0.05	0.05	0.31	0.31	91
6	0.543	1.95	1.09	0.50		0.53	0.48	0.61	0.92	56
8	0.804	2.89	0.94	0.50		1.17	0.64	0.30	1.22	32
10	0.935	3.37	0.48	0.50		1.53	0.36	0.12	1.34	25
12	1.000	3.60	0.23	0.50		1.72	0.19	0.04	1.38	17
Sum			3.60		0.5		1.72	1.38		

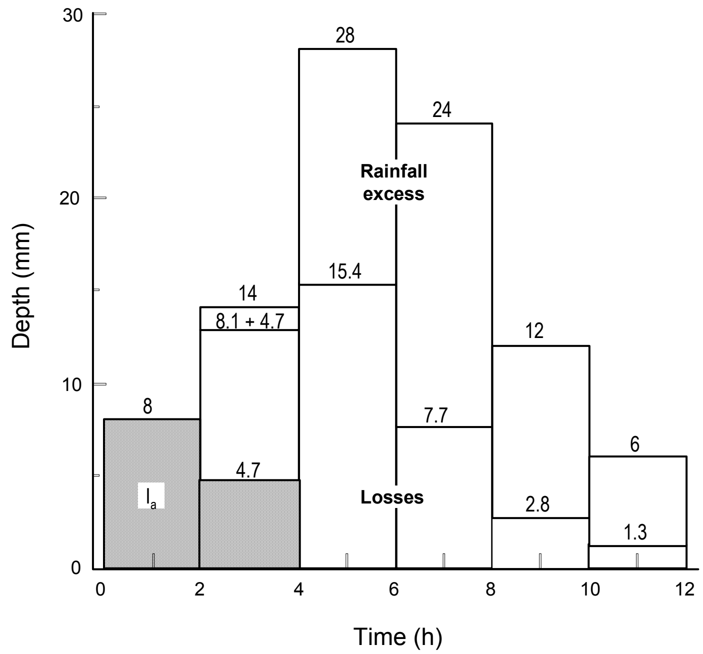


Figure 6.26(SI). Separation of losses and initial abstraction from a design-storm hyetograph using the SCS method.

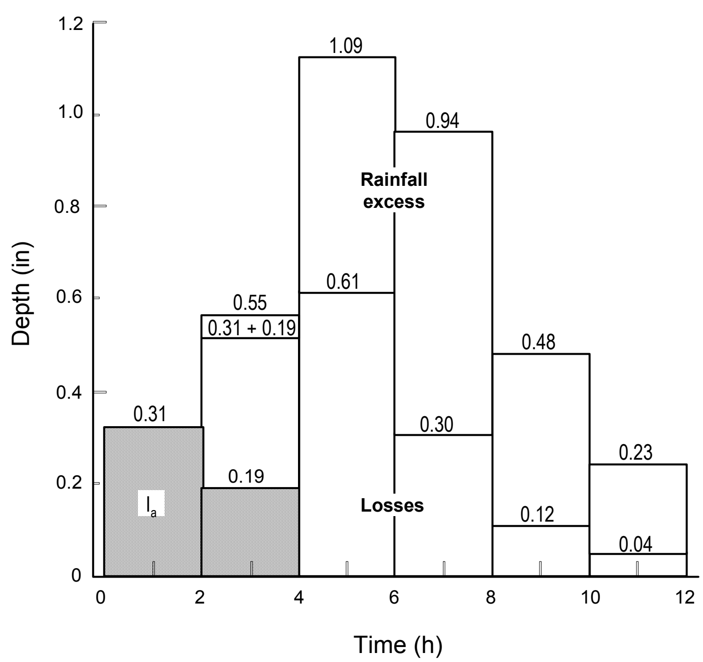


Figure 6.26(CU). Separation of losses and initial abstraction from a design-storm hyetograph using the SCS method.

The rainfall excess for the other cumulative rainfall depths are given in column 7 of Table 6.12. The rainfall excess hyetograph, given in column 8, is computed by subtracting adjacent values of the cumulative rainfall-excess function. The separation of the hyetograph into rainfall excess, initial abstraction, and other losses is shown in Figure 6.26. The proportion of the total rainfall that appears as either losses or initial abstraction is given in column 11 of Table 6.12. The proportion of losses decreases with time while the proportion going to rainfall excess increases.

6.4 OTHER CONSIDERATIONS

6.4.1 Time-Area Unit Hydrographs

Time-area analysis assumes that the drainage area and factors that affect the timing of runoff can be used to develop an S-hydrograph. A time-area diagram is the relationship between runoff travel time and the portion of the watershed that contributes runoff for that travel time. The development of a time-area diagram is best illustrated using an example.

Assume that a watershed consists of six equal subareas, as shown in Figure 6.27a, and that the rainfall hyetograph is as shown in Figure 6.27b. The rainfall duration equals the time of concentration for the watershed and has an intensity i. The distribution of rainfall excess is also shown in Figure 6.27b, with a magnitude of Ci, where C is a runoff coefficient such as that for the rational method; thus the loss function is constant with a magnitude of i(1 - C), and the initial abstraction is assumed to be zero.

Based on the assumption that the rainfall is uniformly distributed over the watershed, the depth of rainfall excess Ci is assumed to fall on each subarea of the watershed. At the end of time $t_c/3$, runoff from only subarea 1 will be appearing at the watershed outlet. Assuming that runoff from subareas 2, 3, and 4 have equal travel times, which equal $2t_c/3$, at the end of time $2t_c/3$ subareas 1 to 4 will be contributing runoff at the outlet. This is shown in the hydrograph of Figure 6.27c. At time $2t_c/3$, rains that fell on subareas 5 and 6 are not contributing to direct runoff at the outlet. At time t_c, all six subareas are contributing to runoff at the outlet, but the storm has reached its duration. During the time interval from t_c to $4t_c/3$, rain that fell during the time interval $t_c/3$ to $2t_c/3$ is still contributing runoff at the outlet from subareas 5 and 6, and rain that fell during the time interval $2t_c/3$ to t_c is contributing runoff from subareas 2, 3, and 4. Thus the runoff ordinate equals 5 units. Rain that fell during the time interval from $2t_c/3$ to t_c on subareas 5 and 6 appears as runoff at the outlet in the time interval $4t_c/3$ to $5t_c/3$. It is also important to observe that the depth of rainfall excess, Cit_c, equals the depth of direct runoff; the depths can be converted to volumes by multiplying by the drainage area, A.

The time-area analysis above is somewhat misleading because the rainfall excess and direct runoff are used with a relatively large, discrete time interval, $t_c/3$. The last particle of rainfall excess falling at the upper end of subarea 5 or 6 at time t_c will require a travel time to the outlet of t_c, which means that it will appear as runoff at the outlet of time $2t_c$. The runoff hydrograph of Figure 6.27c suggests that this particle of rainfall reaches the outlet at $5t_c/3$. The difference is due to the discretizing of the calculations. If the time interval, Δt, goes from $t_c/3$ to an infinitesimally small time, the time-area analysis will yield a hydrograph with a shape similar to that of Figure 6.27f, but with a time base equal to $2t_c$. The peak still equals Ci and occurs at time t_c. This can be shown empirically by using successively smaller time increments. For a time increment of $t_c/6$, which is one-half of the time increment used above, we separate the watershed as shown in Figure 6.27d. This produces the direct runoff hydrograph shown in Figure 6.27e. In this case, the time base of the direct runoff hydrograph is $11t_c/6$; this supports the statement that the time base will approach $2t_c$ as Δt goes to zero.

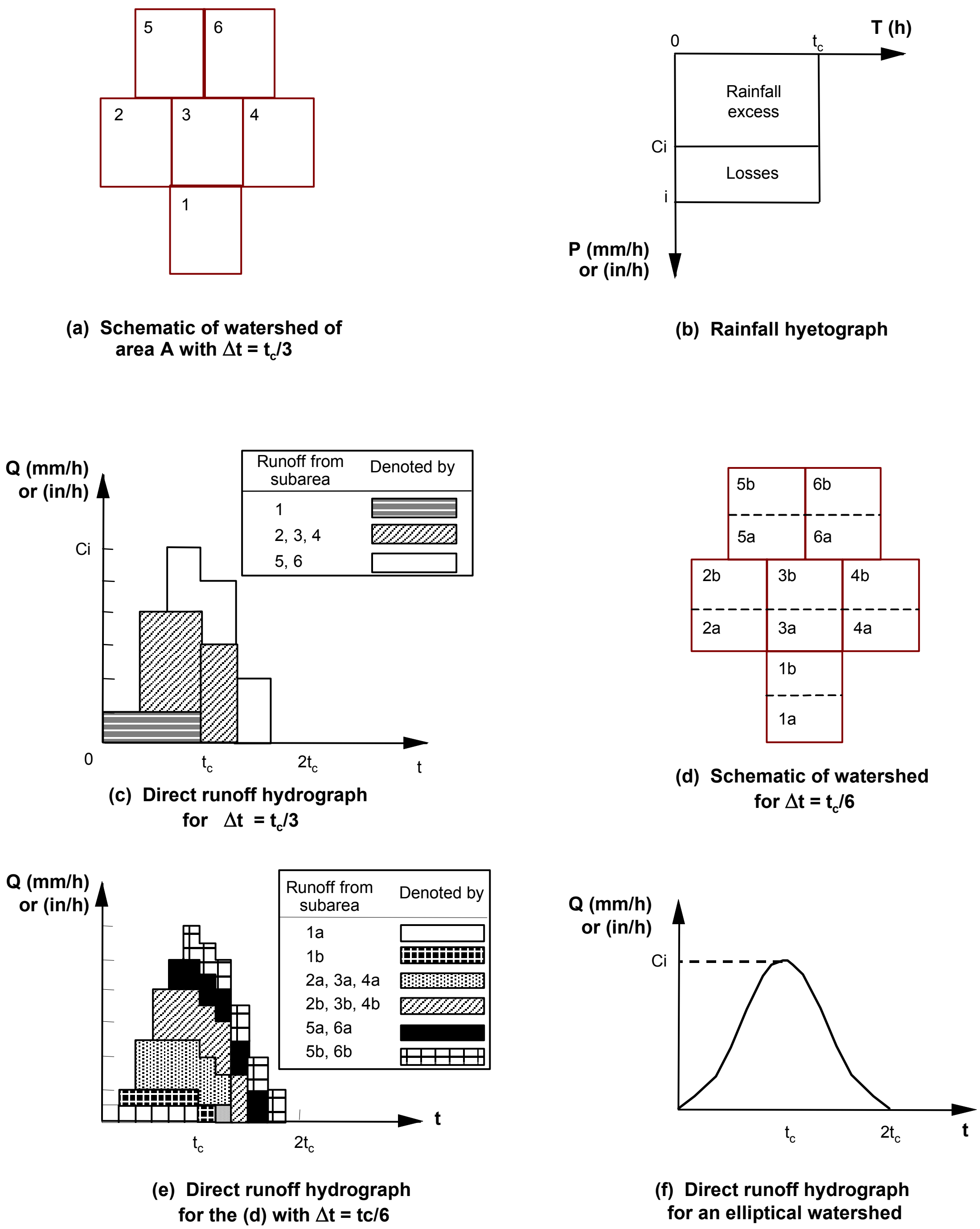


Figure 6.27. Time-area analysis

For a rectangular watershed of length L and width W, the direct runoff hydrograph will have the shape of an isosceles triangle, with a peak Ci and a time base of $2t_c$. Actual watersheds do not have square edges like the schematic in Figure 6.27a, and they are not rectangular. Instead, they appear more elliptical. In such a case, the hydrograph will have a shape such as that shown in Figure 6.27f.

6.4.2 Hydrograph Development Using Assumptions Inherent in the Rational Method

The rational method was previously introduced as a method for estimating peak discharges. The development of the rational method made several assumptions: (1) the rainfall intensity, i, is constant over the storm duration; (2) the rainfall is uniformly distributed over the watershed; (3) the maximum rate of runoff will occur when runoff is being contributed to the outlet from the entire watershed; (4) the peak rate of runoff equals some fraction of the rainfall intensity; and (5) the watershed system is linear.

The same assumptions that underlie the rational formula of Equation 5.12 can be used to develop a hydrograph. One of several possible assumptions can be made to formulate the hydrograph. The easiest solution would be as follows:

1. Estimate the peak discharge of the runoff hydrograph from Equation 5.12.
2. Assume that the runoff hydrograph is an isosceles triangle with a time to peak equal to t_c and a time base $2t_c$.

This method would produce a hydrograph with 50 percent of the volume under the rising limb of the hydrograph and a total volume of $CiAt_c/\alpha$. (α=360 in SI units and α=1 in CU units.) The assumption of an isosceles triangle would probably be reasonable for most design problems on small urban watersheds. To obtain a more realistic description of the runoff hydrograph, the shape of the hydrograph can be determined from the time-area curve, as suggested in the previous section.

Regardless of the assumption about the shape of the hydrograph, the use of a hydrograph based on the rational equation has advantages and disadvantages. Certainly, it is subject to the limitations of the rational equation. However, it is easy to develop, and the accuracy should be sufficient for designs on small, highly urbanized watersheds.

A t_c-h unit hydrograph is inherent in the rational method. Since the depth of the unit hydrograph must equal 1 mm (1 in), the ordinates of the UH can be determined by multiplying each ordinate of the direct runoff hydrograph of the rational equation by the conversion factor K:

$$K = \frac{1}{C\,i\,t_c} \qquad (6.25)$$

in which i and t_c are in mm/h (in/h) and hours, respectively. Thus the peak discharge of the unit hydrograph will be KQ. For example, if C = 0.4, i = 60 mm/h, A = 14 ha, and t_c = 0.25 h, the direct runoff would have a peak discharge of 0.933 m^3/s, the conversion factor K would equal 0.1667, and the peak discharge of the 0.25-h unit hydrograph would be 0.156 $m^3/s/mm$. For a 14-hectare watershed and a UH with a time base of 0.5 hour and a peak discharge of 0.156 m^3/s, the volume is 1 mm.

Similarly in CU units, if C= 0.4, i=2.4 in/h, A=35 ac, and t_c=0.25h, the direct runoff would have a peak discharge of 34 ft^3/s, the conversion factor K would equal 4.167, and the peak discharge of the 0.25-h unit hydrograph would be 142 ft^3/s/in. The volume under this hydrograph is equal to 1 in.

6.4.3 Design Hydrographs by Transposition

Another method that can be used to develop a unit hydrograph at an ungauged 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 estimated by the equation:

$$T_L = \alpha C_t \left(\frac{L L_c}{S^{0.5}} \right)^N \qquad (6.26)$$

where,

T_L = lag time, h
L = length of the longest watercourse, km (mi)
L_c = length along the longest watercourse from the outlet to a point opposite the centroid of the basin, km (mi)
S = slope of the longest watercourse, percent
C_t = basin coefficient determined from hydrologically homogeneous areas
K = exponent determined from hydrologically homogeneous areas and usually equal to 0.33
α = unit conversion constant equal to 0.75 for SI and 1.0 for CU units.

The coefficients in Equation 6.25 and the lag for the ungauged site can be determined from a full logarithmic plot of T_L vs. ($LL_{ca}/S^{0.5}$). 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 checked to ensure it represents 1 mm of runoff from the basin of interest. If not, the shape is adjusted until the volume is reasonably close to 1 mm (1 in).

Often the designer is confronted with a case where stream flow 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 Chapter 7 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 can be used to obtain a design hydrograph.

CHAPTER 7

HYDROGRAPH ROUTING

Two of the more common uses for routing of design hydrographs are to analyze the effects of a channel modification on 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 to calculate outflow from a stream reach given inflow and channel characteristics. Reservoir routing techniques are used to calculate outflow given inflow and storage characteristics. These two techniques are discussed more fully in the following sections.

7.1 CHANNEL ROUTING

Channel 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 due to flow resistance along the channel boundaries and the storage of water in the channel and floodplain. An example of inflow and outflow hydrographs from a stream reach is provided in Figure 7.1. Note that the hydrograph is attenuated and translated as it moves downstream.

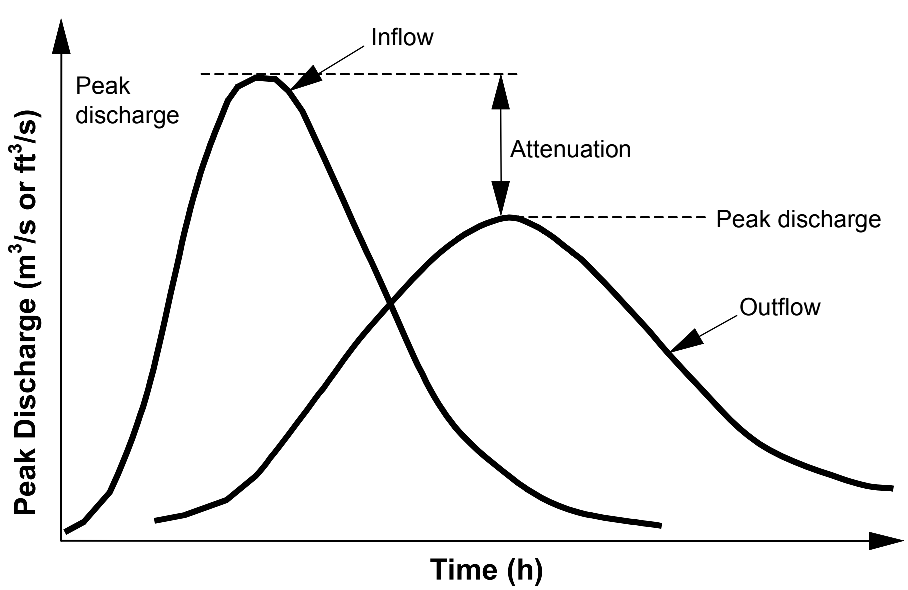


Figure 7.1. Inflow and outflow hydrographs from a stream reach

The general equation for channel routing is based on continuity and represents an accounting of all flow within a reach. Mathematically, the continuity of mass can be written in terms of storage as:

$$\frac{dS}{dt} = I - O \tag{7.1}$$

where,

I = inflow, m^3/s (ft^3/s)
O = outflow, m^3/s (ft^3/s)
t = time, s
S = channel storage, m^3 (ft^3).

Clearly, Equation 7.1 does not account for lateral or tributary inflow.

A number of techniques are available for routing hydrographs through channels. Four commonly used methods are presented in this chapter: Muskingum, kinematic wave, Muskingum-Cunge, and the modified Att-Kin method. The method to choose for a given reach depends on the amount and type of data available, as well as the nature of the hydrograph to be routed. Section 7.1.5 presents an example in which an inflow hydrograph is routed through a stream reach using each of these methods.

7.1.1 Muskingum Routing Method

In the Muskingum routing method, the attenuation of the hydrograph as it moves downstream is assumed to be due to storage within 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 Muskingum method is based upon the assumption that the storage within a given reach of river is given by:

$$S = K\ \left[X\ I + \left(1 - X\right) O\right] \tag{7.2}$$

where,

S = storage, m^3 (ft^3)
I = inflow to the reach, m^3/s (ft^3/s)
O = outflow from the reach, m^3/s (ft^3/s)
K = empirical constant usually set equal to the wave travel time through the reach, s
X = empirical constant that weights the relative importance of inflow versus outflow in determining the storage (varies between 0 and 0.5).

As a first step, the inflow hydrograph is divided into successive time periods, Δt, 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 one routing period. The finite difference form of the continuity equation, Equation 7.1, can be rewritten in terms of the routing period as:

$$\frac{1}{2}\ (I_1 + I_2) - \frac{1}{2}(O_1 + O_2) = \frac{(S_2 - S_1)}{\Delta t} \tag{7.3}$$

If the known quantities of Equation 7.3 are placed on one side of the equal sign and the unknowns on the other side, Equation 7.3 becomes:

$$I_1 + I_2 + \frac{2 S_1}{\Delta t} - O_1 = \left(\frac{2 S_2}{\Delta t} + O_2 \right) \qquad (7.4)$$

where the subscripts 1 and 2 represent values of the parameters at the beginning and ending of a time period. Substituting Equation 7.2 into Equation 7.4 yields the following relation:

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 \qquad (7.5)$$

where

- I_2 = inflow at the end of t, m^3/s (ft^3/s)
- I_1 = inflow at the beginning of t, m^3/s (ft^3/s)
- O_2 = outflow at the end of t, m^3/s (ft^3/s)
- O_1 = outflow at the beginning of t, m^3/s (ft^3/s)

and,

$$C_o = \frac{- K X + 0.5 \Delta t}{K - K X + 0.5 \Delta t} \qquad (7.6)$$

$$C_1 = \frac{K X + 0.5 \Delta t}{K - K X + 0.5 \Delta t} \qquad (7.7)$$

$$C_2 = \frac{K - K X - 0.5 \Delta t}{K - K X + 0.5 \Delta t} \qquad (7.8)$$

$$C_o + C_1 + C_2 = 1 \qquad (7.9)$$

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 pairs of inflow and outflow hydrographs; however, such data are rarely available so more approximate methods are employed.

Values of K and X are determined from data by a trial-and-error process as follows:

1. For each point in time, compute the storage S_2 by rearranging Equation 7.3:

$$S_2 = S_1 + \Delta t \left(\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \right) \qquad (7.10)$$

   S_1 is usually assumed to be zero for the initial condition.

2. Using a trial value of X, compute [X I + (1 - X)O] for each point in time.

3. Plot the computed storage S from Step 1 versus [X I + (1 - X)O] from step 2 for each point in time. This will result in a closed loop.

Revise the value of X and repeat Steps 1 to 3 until the plot shows a minimum amount of deviation from a straight line drawn through the center of the loop.

4. Use the slope of the line as the best estimate of K and the value of X that produced the minimum deviation line in Step 4 as the estimate of X.

Because K and X are calibrated for a specific stream reach, these values are valid only for that particular stream reach for which the calibration data were taken.

When data are not available, K is estimated to be the average travel time through the reach. The discharge used in determining a value for K is the average discharge for the hydrograph. Using Manning's equation to derive an expression for wave speed (celerity) = dQ/dA and assuming a wide channel, then celerity, $c = \beta V$, where V = flow velocity and $\beta = 5/3$.

The value of X must be between 0 and 0.5. If X > 0.5, the hydrograph is amplified as it moves downstream, which does not make physical sense. In the absence of any other data, X is usually assumed to be between 0.2 and 0.3.

7.1.2 Kinematic Wave Method

A kinematic wave is a wave for which inertia and pressure (flow depth) gradient terms are assumed to be negligible compared to the friction and gravity terms (Ponce, 1989). Neglecting these terms from the momentum (or dynamic) equation reduces the dynamic equation to

$$S_o = S_f \tag{7.11}$$

where,

S_o = channel bottom slope
S_f = friction or energy slope.

The equation for a kinematic wave is then derived from the equation of continuity:

$$\frac{\partial Q}{\partial t} + c\,\frac{\partial Q}{\partial x} = 0 \tag{7.12}$$

where,

Q = flow rate
x = distance along the channel bottom
t = time
c = wave celerity = dQ/dA.

Equation 7.12 assumes no lateral inflow. Using Manning's equation to derive an expression for celerity dQ/dA and assuming a wide channel, then $c = \beta V$, where V = flow velocity and $\beta = 5/3$. Three important properties that distinguish kinematic wave routing (Robeson, et al., 1988) are:

- Kinematic waves travel only in the downstream direction.

- The wave shape does not change, and there is no attenuation of the wave height, only translation.

- The wave speed is c = dQ/dA.

The kinematic wave equation (Equation 7.12) is a nonlinear, first-order partial differential equation. It is nonlinear because the celerity is a function of velocity, which varies with discharge. If the nonlinearity is mild, the celerity can be approximated by a constant. The kinematic wave equation can then be discretized using a linear numerical scheme. Using central differences and a simplified form, Equation 7.12 becomes (Ponce, 1989):

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 \tag{7.13}$$

where,

$$C_o = \frac{C - 1}{1 + C} \tag{7.14}$$

$$C_1 = 1 \tag{7.15}$$

$$C_2 = \frac{1 - C}{1 + C} \tag{7.16}$$

and C is the Courant number:

$$C = V \beta \frac{\Delta t}{\Delta x} \tag{7.17}$$

where V is the average channel velocity. The Courant number must be equal or close to 1 to avoid numerical dispersion, which causes errors in the numerical solutions. The solution will vary with the chosen grid size, Δx. Therefore, care must be taken when using the kinematic wave model that the Courant number be as close to 1 as possible, but not greater than 1. This means that if Δt, β, and V are specified, then Δx must be chosen such that the Courant number criterion is satisfied. Since the kinematic wave method can only translate a hydrograph, any attenuation of the inflow hydrograph is produced by numerical dispersion.

The kinematic wave equation is appropriate for steep channels with little or no downstream control. It is not appropriate for mild or flat slopes because significant attenuation of the hydrograph occurs on these slopes, which is not accounted for in the kinematic wave model. Input for the model is primarily in the form of a discharge-area relationship.

7.1.3 Muskingum-Cunge Method

The Muskingum-Cunge routing method has gained popularity in recent years as a method that, while similar to the Muskingum method, does not require extensive hydrologic data for calibration. The method is considered a "hybrid" routing method; it is like hydrologic methods, but contains more physical information typical of hydraulic routing methods. The coefficients are functions of the physical parameters of the channel. The model physically accounts for the diffusion that is present in most natural channels.

The diffusion wave equation is derived from the equations of continuity and momentum. The Muskingum-Cunge method is one method of solution of the diffusion equation. The computational equation is the same as the Muskingum equation (Equation 7.5):

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 \qquad (7.18)$$

However, the computation of C_i differs:

$$C_o = \frac{-1 + C + D}{1 + C + D} \qquad (7.19)$$

$$C_1 = \frac{1 + C - D}{1 + C + D} \qquad (7.20)$$

$$C_2 = \frac{1 - C + D}{1 + C + D} \qquad (7.21)$$

The Courant number, C, is:

$$C = c \frac{\Delta t}{\Delta x} \qquad (7.22)$$

The diffusion coefficient, D, is:

$$D = \frac{Q_o}{c\, S_o\, T\, \Delta x} \qquad (7.23)$$

where,

t = time, s
x = distance along the channel, m (ft)
c = celerity, m/s (ft/s)
Q_o = reference discharge, m^3/s (ft^3/s)
T = top width of channel flow at Q_0, m (ft)
S_o = slope.

Celerity, c, is obtained from a rating curve, c = (dQ/dA). For wide channels, it may be approximated as c = βV, as in the kinematic wave method. The reference discharge is commonly chosen as the average of the peak discharge and base flow of the inflow hydrograph. It is intended to represent hydraulic conditions of the wave. As in the Muskingum method, $C_o + C_1 + C_2 = 1$.

Selection of Δt must at least meet both of the following two criteria in order to capture event detail and avoid numerical dispersion:

- $\Delta t < 0.2\ t_p$
- Δt < wave travel time through the reach (Δx)

After selection of Δt, Δx must be selected so that the Courant number, C, equals 1 or is slightly less. For best results, the sum of C and D should be greater than or equal to 1. It may be noted that C_1 and C_2 can be positive or negative, unlike the Muskingum method.

The Muskingum-Cunge method is appropriate for use on most stream channels. It accounts for diffusion of the flood wave; however, if there are significant backwater effects caused by upstream or downstream controls, then this method should not be used (actually, only the full dynamic equation can account for backwater effects). The main advantage of using the Muskingum-Cunge over the Muskingum routing method is that the Muskingum-Cunge method is physically-based and requires minimal stream flow data. The parameters are based on the rating curve and slope. Therefore, this method is ideal for use in ungauged streams.

7.1.4 Modified Att-Kin Method

The modified Att-Kin method transforms the continuity-of-mass relationship of Equation 7.1 to the following:

$$I_1 - \frac{O_2 + O_1}{2} = \frac{S_2 - S_1}{\Delta t} \tag{7.24}$$

Substituting S = KO into Equation 7.24 and solving for O_2 yields the following:

$$O_2 = \left(\frac{2\Delta t}{2K + \Delta t}\right) I_1 + \left(1 - \frac{2\Delta t}{2K + \Delta t}\right) O_1 \tag{7.25}$$

$$= C_m I_1 + (1 - C_m)\, O_1 \tag{7.26}$$

in which C_m is the routing coefficient for the modified Att-Kin method. The value of K is assumed to be given by:

$$K = \frac{L}{mV} \tag{7.27}$$

in which L is the reach length and V is the velocity, defined by the continuity equation:

$$V = \frac{q}{A} \tag{7.28}$$

in which A is related to q by the rating curve equation:

$$q = x\, A^m \tag{7.29}$$

Equation 7.29 corresponds directly to the stage-discharge relationship $O = ah^b$ since A is a function of h. If the discharge is derived using Manning's equation, then:

$$q = \frac{\alpha}{n} R_h^{2/3} A S^{1/2} = \frac{1.0}{n} S^{1/2}\, A \left(\frac{A}{P}\right)^{2/3} = \frac{S^{1/2}}{n P^{2/3}} A^{5/3} \tag{7.30}$$

α = unit conversion constant equal to 1.0 in SI units and α = 1.49 in CU units. Comparing Equations 7.29 and 7.30 indicates that:

$$m = \frac{5}{3} \qquad (7.31)$$

and

$$x = \frac{S^{1/2}}{n P^{2/3}} \qquad (7.32)$$

Therefore, m is a constant with these assumptions and x is a function of the characteristics of the cross-section.

$$\log q = \log x + m \log A \qquad (7.33)$$

The rating table of Equation 7.29 assumes that the flow (q) and cross-sectional area (A) data measured from numerous storm events will lie about a straight line when plotted on log-log scales. That is, taking the logarithms of Equation 7.29 yields the straight line. Thus the intercept is log x, and m is the slope of the line. For the form of Equation 7.33, the intercept log x equals the logarithm of the discharge at an area of 1.0.

The coefficients of Equation 7.33 can be fit with any one of several methods. Visually, a line could be drawn through the points and the slope computed; the value of x would equal the discharge for the line when A equals 1.0.

The coefficients of Equation 7.33 could also be fitted using linear regression analysis after making the logarithmic transform of the data. This is identical to the analysis of the rating-table fitting in which stage is the predictor variable. The statistical fit may be more rational than the visual fit, especially when the scatter of the data is significant and the visual fit is subject to a lack of consistency.

As indicated before, Manning's equation can be used where rating table data (i.e., q versus A) are not available. Manning's equation can be applied for a series of depths and the rating table constructed. Of course, the rating table values and, thus, the coefficients are dependent on the assumptions underlying Manning's equation.

In many cases, the graph of log q versus log A will exhibit a nonlinear trend, which indicates that the model of Equation 7.29 is not correct. The accuracy in using Equation 7.29 to represent the rating table will depend on the degree of nonlinearity in the plot. The SCS TR-20 manual (SCS, 1984) provides a means of deriving a weighted value of m, which is as follows. The slope between each pair of points on the rating curve is estimated numerically:

$$S_i = \frac{\log q_i - \log q_{i-1}}{\log A_i - \log A_{i-1}} \qquad (7.34)$$

in which S_i is the slope between points i and (i-1). The weighted value of m, which is denoted as $\bar{m}$, is:

$$\overline{m} = \frac{q_3 S_3 + \sum_{j=4}^{k} (q_j - q_{j-1}) S_j}{q_i} \qquad (7.35)$$

in which k is the number of pairs of points on the rating table. The weighting of Equation 7.35 provides greater weight to the slope between points for which the range of (q_j-q_{j-1}) is larger.

The modified Att-Kin method uses Equation 7.26 to perform the routings necessary to derive the downstream hydrograph. The modified Att-Kin method provides for both attenuation and translation. To apply the modified Att-Kin method, the values of m and x in Equation 7.29 are evaluated using either cross-section data or a rating table developed from measured runoff events. The value of K is then computed from Equation 7.27 and used to compute C_m. The routing equation (Equation 7.26) can then be used to route the upstream hydrograph.

After deriving the first estimate of the downstream hydrograph, it is necessary to check whether or not hydrograph translation is necessary. The downstream (routed) hydrograph computed with Equation 7.26 is further translated when the kinematic travel time (Δt_p) is greater than the difference in the times to peak between the upstream hydrograph and the computed downstream hydrograph. This time difference between the upstream and downstream hydrographs peaks is denoted as:

$$\Delta t_{ps} = t_{po} - t_{pi} \qquad (7.36)$$

in which t_{po} and t_{pi} are the times-to-peak of the downstream (outflow) and upstream (inflow) hydrographs, respectively. The kinematic travel time is given by:

$$\Delta t_p = \frac{S_{po}}{q_{po}} \left[\frac{(q_I / q_{po})^{1/m} - 1}{(q_I / q_{po}) - 1} \right] / 3600 \qquad (7.37)$$

in which q_{po} is the peak discharge of the downstream hydrograph, q_I, is the peak discharge of the upstream hydrograph, and S_{po} is given by:

$$S_{po} = \left(\frac{q_{po}}{k} \right)^{1/m} \qquad (7.38)$$

and

$$k = \frac{x}{L^m} \qquad (7.39)$$

If $\Delta t_p > \Delta t_{ps}$, the storage-routed hydrograph from Equation 7.26 is translated by an amount (Δt_p-Δt_{ps}).

This procedure can be summarized by the following steps:

1. From cross-section information, evaluate the rating table coefficients m and x.

2. Compute K and then C_m. (Note: It is necessary for $C_m < 1$ and preferable that $C_m < 0.67$.)

3. Use the routing equation (Equation 7.26) to route the upstream hydrograph.

4. Compute Δt_{ps} from Equation 7.36.

5. Compute the kinematic travel time Δt_p (Equation 7.37).

6. If $\Delta t_p > \Delta t_{ps}$, translate the computed downstream hydrograph.

7.1.5 Application of Routing Methods

Example 7.1(SI). Consider a river reach as shown in Figure 7.2. A hydrograph developed at point A is to be routed along the 4.8 km reach of river. What effect will this channel routing have on the peak discharge experienced at the roadway at point B? Four routing methods – Muskingum, Kinematic Wave, Muskingum-Cunge, and Modified Att-Kin are applied and compared.

A synthetic hydrograph is developed at Point A using the procedures presented in Chapter 6 for a 25-year design discharge. The upstream hydrograph, which is used as the inflow, is given in Table 7.1(SI). The peak discharge is 84 m^3/s. The routing coefficients are assumed constant and based on the reference discharge for this hydrograph at 34 m^3/s. Using the given data and the trapezoidal cross-section given in Figure 7.2, the following values can be calculated for this discharge:

depth = 2 m
cross-sectional area = 24 m^2
average velocity = 1.4 m/s
celerity, c = (5/3) V = 2.33 m/s
wave travel time = 4800 m/(2.33 m/s (3600 s/h)) = 0.57 hours.

Muskingum Method

For the Muskingum method, the coefficients C_o, C_1, and C_2 are first computed from Equations 7.6, 7.7, and 7.8 using Δt = 0.5 hour and assumed values of X = 0.2 and K = 0.57 hour as follows:

$$C_o = \frac{-0.57(0.2) + 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.193$$

$$C_1 = \frac{0.57(0.2) + 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.516$$

$$C_2 = \frac{0.57 - 0.57(0.2) - 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.291$$

The sum of the coefficients is $C_o + C_1 + C_2$ = 0.193 + 0.516 + 0.291 = 1.000. The outflow hydrograph ordinates can now be computed with Equation 7.5. Beginning at t = 0.5 hours:

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1$$
$$= 0.193(7) + 0.516(0) + 0.291(0) = 1.4$$

At t = 1 hour:

$$O_2 = 0.193(13) + 0.516(7) + 0.291(1.4) = 6.5$$

These values along with the remaining calculations are tabulated in Table 7.1(SI).

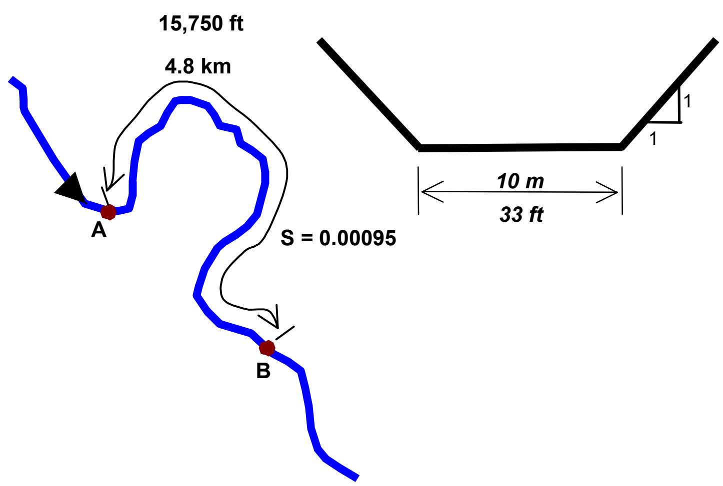


Figure 7.2. Schematic of river reach for Example 7.1

Table 7.1(SI). Inflow and Outflow Hydrographs for Selected Routing Methods

Time (h)	Inflow (m^3/s)	Muskingum Outflow (m^3/s)	Kinematic Wave Outflow (m^3/s)	Muskingum-Cunge Outflow (m^3/s)	Modified Att-Kin Outflow (m^3/s)
0.0	0	0.0	0.0	0.0	0.0
0.5	7	1.4	0.0	0.4	0.0
1.0	13	6.5	6.2	6.2	4.3
1.5	23	13.0	11.9	12.4	9.6
2.0	32	21.9	21.7	21.7	17.8
2.5	49	32.4	30.3	31.2	26.4
3.0	68	47.9	46.6	47.0	40.2
3.5	76	63.7	66.1	64.9	57.1
4.0	84	74.0	74.9	74.6	68.6
4.5	78	80.0	83.8	82.1	78.0
5.0	71	77.3	78.8	78.3	78.0
5.5	60	70.8	72.2	71.6	73.7
6.0	52	61.7	61.3	61.5	65.4
6.5	46	53.7	53.0	53.3	57.2
7.0	40	47.1	46.8	46.9	50.4
7.5	36	41.4	40.7	40.9	44.1
8.0	32	36.8	36.6	36.6	39.2
8.5	28	32.7	32.5	32.6	34.8
9.0	24	28.6	28.5	28.5	30.7
9.5	20	24.6	24.5	24.5	26.6
10.0	16	20.6	20.5	20.5	22.6
10.5	13	16.8	16.5	16.6	18.6
11.0	11	13.7	13.4	13.5	15.2
11.5	7	11.0	11.4	11.2	12.6
12.0	6	8.0	7.3	7.6	9.2
12.5	3	6.0	6.3	6.1	7.3
13.0	0	3.3	3.4	3.4	4.7
13.5	0	1.0	0.2	0.6	1.8
14.0	0	0.3	0.0	0.1	0.7
14.5	0	0.1	0.0	0.0	0.3
15.0	0	0.0	0.0	0.0	0.1
15.5	0	0.0	0.0	0.0	0.0
16.0	0	0.0	0.0	0.0	0.0
16.5	0	0.0	0.0	0.0	0.0
17.0	0	0.0	0.0	0.0	0.0

Kinematic Wave Method

The same inflow hydrograph can be routed through the 4.8 km reach using the kinematic wave method. If the reference discharge and the corresponding cross-sectional area are used to compute the velocity and celerity, V = 1.4 m/s. Assuming β = 5/3 and Δt = 1,800 s, then C = 0.88, C_o = -0.064, C_1 = 1, and C_2 = 0.064. The resulting outflow hydrograph is computed from Equation 7.13 and is shown in Table 7.1(SI). The calculations proceed in the same manner as for the Muskingum method. Beginning at t = 0.5 hour:

$$O_2 = -0.064(7) + 1(0) + 0.064(0) = -0.4 \ m^3/s$$

Since a negative flow is not possible, a value of zero is assumed. At t = 1 hour:

$$O_2 = -0.064(13) + 1(7) + 0.064(0) = 6.2 \ m^3/s$$

Note that the hydrograph has been translated (i.e., the peak discharge now occurs at hour 4.5), but has not attenuated.

Muskingum-Cunge Method

The inflow hydrograph can also be routed using the Muskingum-Cunge method. From Equations 7.19 through 7.23, and using the same Δx and Δt as for the kinematic wave method, C = 0.88, D = 0.23, C_o = 0.052, C_1 = 0.782, and C_2 = 0.166. The outflow hydrograph is computed from Equation 7.18 and is given in Table 7.1(SI) in a manner similar to the Muskingum and kinematic wave methods. The peak flow attenuates to 82.1 m^3/s, and translates to hour 4.5.

Modified Att-Kin Method

The inflow hydrograph was again routed using the modified Att-Kin method. Using Equation 7.27, K=0.57 h from which the routing coefficient, C_m , is calculated to equal 0.609. Using the routing equation, the downstream hydrograph is given in Table 7.1(SI). The peak outflow is 78.0 m^3/s and has been translated 1 hour later to hour 5.0.

Equation 7.37 must now be applied to determine if further hydrograph translation is required. Δt_p = 0.4 h which is less than the 1-hour translation shown in Table 7.1(SI); therefore, no further translation is required.

Example 7.1(CU). Consider a river reach as shown in Figure 7.2. A hydrograph developed at point A is to be routed along the 15,750 ft reach of river. What effect will this channel routing have on the peak discharge experienced at the roadway at point B? Four routing methods – Muskingum, Kinematic Wave, Muskingum-Cunge, and Modified Att-Kin are applied and compared.

A synthetic hydrograph is developed at Point A using the procedures presented in Chapter 6 for a 25-year design discharge. The upstream hydrograph, which is used as the inflow, is given in Table 7.1(CU). The peak discharge is 3,090 ft^3/s. It is assumed that the routing coefficients are constant and based on the reference discharge for this hydrograph at 1,200 ft^3/s. Using the given data and the trapezoidal cross-section given in Figure 7.2, the following values can be calculated for this discharge:

depth = 6.6 ft
cross-sectional area = 261 ft^2
average velocity = 4.6 ft/s
wave velocity (celerity) = (5/3) V = 7.7 ft/s
wave travel time = 15,750 ft / [7.7 ft/s (3600 s/h)] = 0.57 hour.

Muskingum Method

For the Muskingum method, the coefficients C_o, C_1, and C_2 are first computed from Equations 7.6, 7.7, and 7.8 using Δt = 0.5 hour and assumed values of X = 0.2 and K = 0.57 hour as follows:

$$C_o = \frac{-0.57(0.2) + 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.193$$

$$C_1 = \frac{0.57(0.2) + 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.516$$

$$C_2 = \frac{0.57 - 0.57(0.2) - 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.291$$

The sum of the coefficients is $C_o + C_1 + C_2 = 0.193 + 0.516 + 0.291 = 1.000$. The outflow hydrograph ordinates can now be computed with Equation 7.5. Beginning at t = 0.5 hours:

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1$$
$$= 0.193(247) + 0.516(0) + 0.291(0) = 48\ ft^3/s$$

At t = 1 hour:

$$O_2 = 0.193(459) + 0.516(247) + 0.291(48) = 230\ ft^3/s$$

These values along with the remaining calculations are tabulated in Table 7.1(CU).

Table 7.1(CU). Inflow and Outflow Hydrographs for Three Routing Methods

Time (h)	Inflow (ft^3/s)	Muskingum Outflow (ft^3/s)	Kinematic Wave Outflow (ft^3/s)	Muskingum -Cunge Outflow (ft^3/s)	Modified Att-Kin Outflow (ft^3/s)	Muskingum Outflow- Modified Channel (ft^3/s)
0.0	0	0	0	0	0	0
0.5	247	48	0	13	0	26
1.0	459	230	218	219	150	173
1.5	812	461	421	438	338	372
2.0	1,130	772	767	766	627	655
2.5	1,730	1142	1,068	1,101	933	987
3.0	2,401	1690	1,645	1,660	1,418	1,478
3.5	2,684	2250	2,334	2,293	2,017	2,031
4.0	2,966	2614	2,644	2,634	2,423	2,430
4.5	2,754	2825	2,959	2,900	2,754	2,712
5.0	2,507	2730	2,783	2,765	2,754	2,710
5.5	2,119	2500	2,549	2,530	2,604	2,555
6.0	1,836	2178	2,165	2,172	2,308	2,278
6.5	1,624	1897	1,871	1,881	2,021	2,005
7.0	1,412	1664	1,653	1,656	1,779	1,767
7.5	1,271	1460	1,436	1,445	1,556	1,551
8.0	1130	1300	1,291	1,293	1,382	1,378
8.5	989	1154	1,149	1,150	1,229	1,223
9.0	847	1011	1,008	1,008	1,083	1,075
9.5	706	868	866	866	939	931
10.0	565	727	725	725	797	789
10.5	459	592	582	586	656	651
11.0	388	485	471	476	536	535
11.5	247	389	402	395	446	437
12.0	212	282	259	270	325	326
12.5	106	212	222	216	256	250
13.0	0	117	120	119	165	157
13.5	0	34	8	20	64	68
14.0	0	10	0	3	25	30
14.5	0	3	0	1	10	13
15.0	0	1	0	0	4	6
15.5	0	0	0	0	2	2
16.0	0	0	0	0	1	1
16.5	0	0	0	0	0	0
17.0	0	0	0	0	0	0

Kinematic Wave Method

The same inflow hydrograph can be routed through the reach using the kinematic wave method. If the reference discharge and the corresponding cross-sectional area are used to compute the velocity and celerity, then V = 4.6 ft/s. Assuming β = 5/3 and Δt = 1,800 s, then C = 0.88, C_o = -0.064, C_1 = 1, and C_2 = 0.064. The resulting outflow hydrograph is computed from Equation 7.13 and is shown in Table 7.1(CU). The calculations proceed in the same manner as for the Muskingum method. Beginning at t = 0.5 hour:

$$O_2 = -0.064(247) + 1(0) + 0.064(0) = -15\ ft^3/s$$

Since a negative flow is not possible, a value of zero is assumed. At t = 1 hour:

$$O_2 = -0.064(459) + 1(247) + 0.064(0) = 218\ ft^3/s$$

Note that the hydrograph has been translated (i.e., the peak discharge now occurs at hour 4.5), but has not attenuated.

Muskingum-Cunge Method

The inflow hydrograph can also be routed using the Muskingum-Cunge method. From Equations 7.19 through 7.23, and using the same Δx and Δt as for the kinematic wave method, C = 0.88, D = 0.23, C_o = 0.052, C_1 = 0.782, and C_2 = 0.166. The outflow hydrograph is computed from Equation 7.18 and is given in Table 7.1(CU) in a manner similar to the Muskingum and kinematic wave methods. The peak flow attenuates to 2,900 ft^3/s, and translates to hour 4.5.

Modified Att-Kin Method

The inflow hydrograph was again routed using the modified Att-Kin method. Using Equation 7.27, K = 0.57 h from which the routing coefficient, C_m , is calculated to equal 0.609. Using the routing equation, the downstream hydrograph is given in Table 7.1(CU). The peak outflow is 2,754 ft^3/s and has been translated from 1 hour to hour 5.0.

Equation 7.37 must now be applied to determine if further hydrograph translation is required. Δt_p = 0.5 h which is less than the 1-hour translation shown in Table 7.1(CU); therefore, no further translation is required.

7.2 RESERVOIR ROUTING

Whenever the outflow from a river channel section or body of water is dependent only upon the storage in the reach or reservoir, storage routing techniques can be applied. In highway drainage design, this condition is often approximated when water is backed up by a culvert and impounded (stored) by the highway embankment. Another application is in the design of detention storage basins that are often used to mitigate the increase in peak discharge associated with urbanization.

7.2.1 Required Functions for Storage Routing

The method of reservoir routing presented in this section is the Storage-Indication method and is again based on the steady-state continuity equation. Storage routing requires the development of four functions:

1. stage-storage relationship (h vs. S)
2. stage-discharge relationship (h vs. O)
3. storage-discharge relationship (O vs. S)
4. storage-indication relationship (O vs. S/ Δt + O/2)

The stage-storage-discharge (SSD) relationship is formed from the stage-storage and stage-discharge relationships and is a function of both the topography at the site of the storage structure and the characteristics of the outlet facility. The topographic features of the site control the relationship between stage and storage, and the relationship between stage and discharge is primarily a function of the characteristics of the outlet facility. If the same values of stage are used to derive these two relationships, then the corresponding values of storage and discharge can be used to form the storage-discharge relationship.

7.2.2 The Storage-Indication Curve

To form the fourth relationship, the storage-indication curve, Equation 7.3 is algebraically transformed so that the knowns (I_1, I_2, S_1, and O_1) are on one side of the equation and the unknowns (S_2 and O_2) are on the other side:

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \qquad (7.40)$$

The right-hand side of Equation 7.40 can be generalized, with the storage-indication relationship being graphed as O vs. (S/Δt + O/2). The following procedure can be used to develop the storage-indication curve:

1. Select a value of O.
2. Determine the corresponding value of S from the storage- discharge relationship.
3. Use the values of S and O to compute (S + OΔt/2).
4. Plot a point on the storage-indication curve O versus (S + OΔt/2).

Repeat these four steps for a sufficient number of values of O to define the storage-indication curve. Generally, linear interpolation is applied when routing with the storage-indication method. Therefore, values of O should be selected at an interval that is sufficiently small to give good definition to the inflow hydrograph. As a guide, values of O only as large as the peak of the inflow hydrograph are necessary since the ordinates of the outflow hydrograph will not exceed those of the inflow hydrograph.

To avoid numerical instabilities in the computations, the time step, Δt, must be chosen so that at all times:

$$\frac{\Delta O}{2} \leq \frac{\Delta S}{\Delta t} \qquad (7.41)$$

where ΔO and ΔS are the changes in the outflow and storage during the time step. This can be verified graphically by plotting a line of equal values (slope = 1) on the storage-indication curve. If Equation 7.41 is true for all values, the slope of the storage-indication curve will always be less than the slope of the line of equal values. If not true, a smaller time step is required.

The time step should also provide sufficient event detail to accurately model the inflow hydrograph. Therefore, a potential time increment may be $\Delta t < 0.2$ tp. Δt should meet both the criteria of being below the line of equal values and having an appropriate time increment.

7.2.3 Input Requirements for the Storage-Indication Method

The objective of the storage-indication method is to derive the outflow hydrograph. Five elements of data are needed:

1. storage-discharge relationship
2. storage-indication relationship
3. inflow hydrograph
4. initial values of the outflow rate (O_1) and storage (S_1)
5. routing interval (Δt)

While the outflow hydrograph is the primary output for most design problems, the storage function (i.e., S vs. t) is also an important output of the routing method. The maximum value of S from the S-versus-t relationship is the required storage volume at maximum flow stage. The maximum storage occurs when the outflow rate first exceeds the inflow rate.

7.2.4 Computational Procedure

The procedure for routing the inflow hydrograph is as follows:

1. Determine the storage-discharge curve.
2. Select time step, (Δt).
3. Calculate storage-indication curve.
4. Discretize inflow hydrograph.
5. Assume an initial value for O1 and S1 (usually zero or equal to the inflow at the same time).
6. Compute the storage indication value, S1//Δt + O1/2.
7. Use Equation 7.40 to determine the value of S2 /Δ t + O2/2.
8. Obtain O2 from the storage-indication curve.
9. Use O2 with the storage-discharge relationship to obtain S2.

Steps 7 through 9 are repeated for the next time increment using I_2, O_2, and S_2 as the new values of I_1, O_1, and S_1, respectively.

<u>Example 7.2(SI)</u>. A highway engineer needs to design a culvert so that, when the 50-year peak discharge is impounded, the maximum water level is 0.3 m below the roadway elevation. What size CMP culvert should be specified?

The hydrograph associated with the 50-year peak discharge is given in Table 7.2(SI). The depth-discharge relationships for 600-mm and 900-mm CMP culverts are tabulated in columns 1, 3, and 4 of Table 7.3(SI). When the depth is greater than 1.8 meters, the embankment is overtopped and the discharge increases significantly as the embankment begins to function as a broad crested weir. At a depth of 2.1 meters, the discharge is 2.5 m^3/s due to overtopping alone.

The depth-storage relationship is site-specific. For the particular location in this example, the depth-storage relationship is tabulated in columns 1 and 2 of Table 7.3(SI). Using the data in Table 7.3(SI), the values of ($S/\Delta t + O/2$) for the various culvert sizes and a range of values of O are determined. Note that an appropriate value for Δt must be selected. In this example, it is 0.5 h. The storage-indication values determined above are then plotted versus O as shown in Figure 7.3(SI). For the range shown, the storage-indication curves for both culverts show slopes less than the line of equal values except for the 900-mm culvert curve from 0.5 to 1.0 m^3/s. In this range, the computations should be checked for instabilities. If higher slopes appear more prevalent, a smaller time step would be required. The steps of the procedure outlined above are then used to route the inflow hydrograph.

The inflow hydrograph is first routed for the 600-mm diameter culvert in Table 7.4(SI). This table shows a peak outflow of 1.44 m^3/s and a peak storage of 3,969. Since the design objective is not to exceed a depth of 1.5 m (storage = 2,100 m^3), this culvert is inadequate. (Recall that it is desirable to keep the depth below 1.5 meters or 0.3 meter below the embankment elevation.) The same routing procedure is now applied for the 900-mm diameter culvert, which is shown in Table 7.5(SI). For the 900-mm diameter culvert, a peak flow of 1.52 m^3/s is obtained, which can be handled with a depth less than 1.5 meters. A culvert diameter of 900 mm meets the design criteria that the maximum water level remains 0.3 m below the roadway elevation.

Table 7.2(SI). Inflow Hydrograph for CMP Culvert Storage Routing Example

Time (h)	Discharge (m^3/s)
0.0	0.00
0.5	0.30
1.0	0.60
1.5	0.85
2.0	1.10
2.5	1.40
3.0	1.70
3.5	1.40
4.0	1.10
4.5	0.85
5.0	0.60
5.5	0.30
6.0	0.00

Table 7.3(SI). Depth-Storage and Depth-Discharge Relationships

Depth (m)	Storage (m^3)	Discharge- 600 mm Culvert (m^3/s)	Discharge- 900 mm Culvert (m^3/s)
0	0	0	0
0.3	200	0.12	0.17
0.6	500	0.36	0.51
0.9	900	0.57	0.99
1.2	1,400	0.74	1.42
1.5	2,100	0.88	1.73
1.8	3,400	0.99	1.98
1.9	4,000	1.46	2.45
2.0	4,700	2.33	3.32
2.1	5,400	3.45	4.44

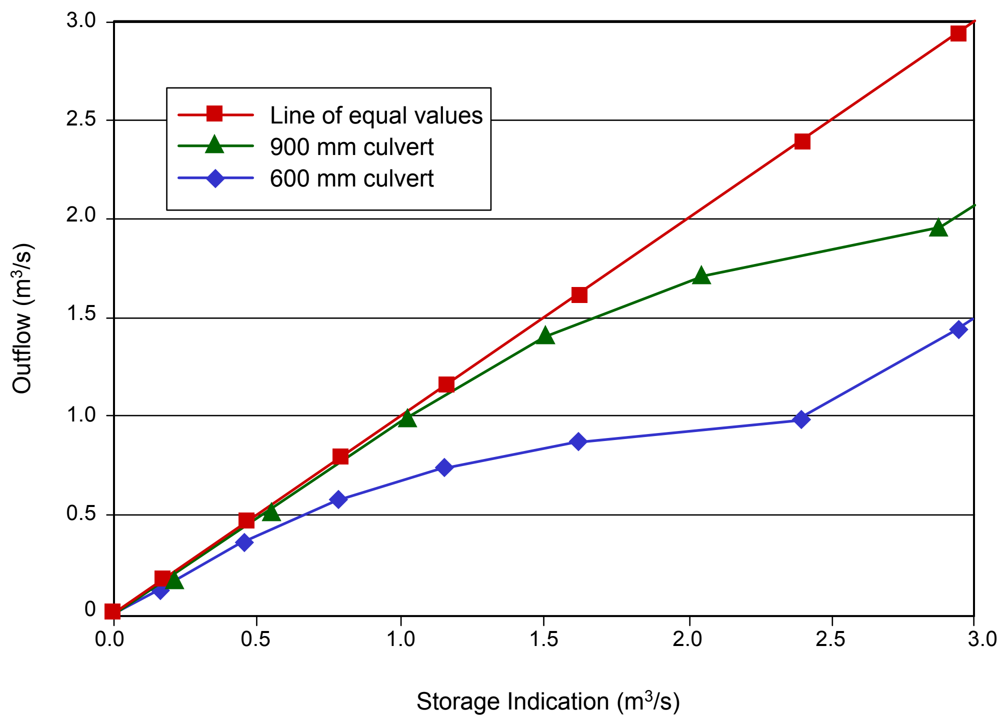


Figure 7.3(SI). Storage-indication curves for Example 7.2

Table 7.4(SI). Hydrograph Routed Through 600-mm Culvert

Time (h)	Inflow (m^3/s)	Average Inflow (m^3/s)	$S/\Delta t + O/2$ (m^3/s)	O (m^3/s)	S (m^3)
0.0	0.00		0.00	0.00	0
		0.15			
0.5	0.30		0.15	0.11	175
		0.45			
1.0	0.60		0.49	0.38	545
		0.73			
1.5	0.85		0.84	0.59	970
		0.98			
2.0	1.10		1.22	0.76	1,506
		1.25			
2.5	1.40		1.71	0.89	2,266
		1.55			
3.0	1.70		2.36	0.99	3,363
		1.55			
3.5	1.40		2.93	1.44	3,969
		1.25			
4.0	1.10		2.74	1.28	3,769
		0.98			
4.5	0.85		2.43	1.03	3,446
		0.73			
5.0	0.60		2.13	0.95	2,969
		0.45			
5.5	0.30		1.62	0.88	2,127
		0.15			
6.0	0.00		0.89	0.62	1,045

Table 7.5(SI). Hydrograph Routed Through 900-mm Culvert

Time (h)	Inflow (m^3/s)	Average Inflow (m^3/s)	$S/\Delta t + O/2$ (m^3/s)	O (m^3/s)	S (m^3)
0.0	0.00		0.00	0.00	0
		0.15			
0.5	0.30		0.15	0.13	153
		0.45			
1.0	0.60		0.47	0.45	444
		0.73			
1.5	0.85		0.75	0.73	687
		0.98			
2.0	1.10		0.99	0.98	895
		1.25			
2.5	1.40		1.26	1.22	1,164
		1.55			
3.0	1.70		1.59	1.48	1,529
		1.55			
3.5	1.40		1.66	1.52	1,623
		1.25			
4.0	1.10		1.39	1.34	1,303
		0.98			
4.5	0.85		1.03	1.02	936
		0.73			
5.0	0.60		0.73	0.72	675
		0.45			
5.5	0.30		0.47	0.44	440
		0.15			
6.0	0.00		0.17	0.15	177

Example 7.2(CU). A highway engineer needs to design a culvert so that when the 50-year peak discharge is impounded, the maximum water level is 1 ft below the roadway elevation. What size CMP culvert should be specified?

The hydrograph associated with the 50-year peak discharge is given in Table 7.2(CU). The depth-discharge relationships for 24-in and 36-in CMP culverts are tabulated in columns 1, 3, and 4 of Table 7.3(CU). When the depth is greater than 6 ft, the embankment is overtopped and the discharge increases significantly as the embankment begins to function as a broad crested weir. At a depth of 6.9 ft, the discharge is 87 ft^3/s due to overtopping alone.

The depth-storage relationship is site-specific. For the particular location in this example, the depth-storage relationship is tabulated in columns 1 and 2 of Table 7.3(CU). Using the data in Table 7.3(CU), the values of ($S/\Delta t + O/2$) for the various culvert sizes and a range of values of O are determined. Note that an appropriate value for Δt must be selected. In this example, it is 0.5 h. The ($S/\Delta t + O/2$) values determined above are then plotted versus O as shown in Figure 7.3(CU). For the range shown, the storage indication curves for both culverts show slopes less than the line of equal values except for the 36-in culvert curve from 20 to 40 ft^3/s. In this range, the computations should be checked for instabilities. If higher slopes had appeared more prevalent, a smaller time step would be required. The steps of the procedure outlined above are then used to route the inflow hydrograph.

The inflow hydrograph is first routed for the 24-in diameter culvert in Table 7.4(CU). This table shows a peak outflow of 51 ft^3/s and a peak storage of 139,000 ft^3/s. Since the design objective is not to exceed a depth of 5 ft (storage = 74,000 ft^3), this culvert is inadequate. (Recall that it is desirable to keep the depth below 5 ft or 1 ft below the embankment elevation.) The same routing procedure is now applied for the 36-in diameter culvert, which is shown in Table 7.5 (CU). For the 36-in diameter culvert, a peak flow of 53 ft^3/s is obtained, which can be handled with a depth less than 5 feet.

Table 7.2(CU). Inflow Hydrograph for CMP Culvert Storage Routing Example

Time (h)	Discharge (ft^3/s)
0	0
0.5	11
1.0	21
1.5	30
2.0	39
2.5	49
3.0	60
3.5	49
4.0	39
4.5	30
5.0	21
5.5	11
6.0	0

Table 7.3(CU). Depth-Storage and Depth-Discharge Relationships

Depth (ft)	Storage (ft^3)	Discharge for 24-inch Culvert (ft^3/s)	Discharge for 36-inch Culvert (ft^3/s)
0.0	0	0	0
1.0	7,000	4	6
2.0	18,000	13	18
3.0	32,000	20	35
4.0	49,000	26	50
5.0	74,000	31	61
6.0	120,000	35	70
6.3	141,000	52	87
6.6	166,000	82	117
6.9	191,000	122	157

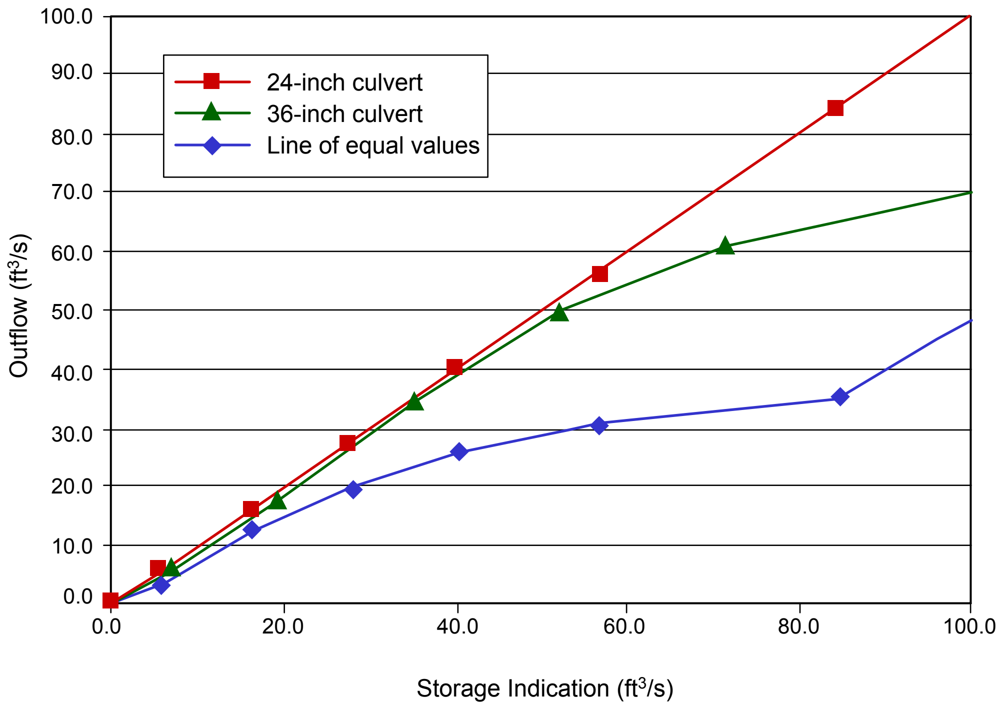


Figure 7.3 (CU). Storage-indication curves for Example 7.2

Table 7.4(CU). Hydrograph Routed Through 24-inch Culvert

Time (h)	Inflow (ft^3/s)	Average Inflow (ft^3/s)	$S/\Delta t + O/2$ (ft^3/s)	O (ft^3/s)	S (ft^3)
0.0	0		0	0	0
		5.5			
0.5	11		6	4	6,540
		16.0			
1.0	21		18	14	19,600
		25.5			
1.5	30		29	21	34,300
		34.5			
2.0	39		43	27	53,500
		44.0			
2.5	49		60	32	80,100
		54.5			
3.0	60		83	35	118,000
		54.5			
3.5	49		103	51	139,000
		44.0			
4.0	39		96	45	132,000
		34.5			
4.5	30		86	36	121,000
		25.5			
5.0	21		75	34	105,000
		16.0			
5.5	11		57	31	75,000
		5.5			
6.0	0		32	22	37,300

Table 7.5(CU). Hydrograph Routed Through 36-inch Culvert

Time (h)	Inflow (ft^3/s)	Average Inflow (ft^3/s)	$S/\Delta t + O/2$ (ft^3/s)	O (ft^3/s)	S (ft^3)
0.0	0		0	0	0
		5.5			
0.5	11		6	5	5,590
		16.0			
1.0	21		17	16	15,900
		25.5			
1.5	30		26	26	24,400
		34.5			
2.0	39		35	35	31,900
		44.0			
2.5	49		44	43	41,000
		54.5			
3.0	60		56	52	54,000
		54.5			
3.5	49		58	53	57,000
		44.0			
4.0	39		49	47	46,000
		34.5			
4.5	30		36	36	33,000
		25.5			
5.0	21		26	25	24,000
		16.0			
5.5	11		17	16	15,900
		5.5			
6.0	0		6	6	6,600

CHAPTER 8

DETENTION POND ANALYSIS AND DESIGN

It is widely recognized that land development, especially in urban areas, is responsible for significant changes in runoff characteristics. Within the context of the hydrologic cycle, land development generally decreases the natural storage of a watershed. The removal of trees and vegetation reduces the volume of interception storage. Grading of the site reduces the volume of depression storage and often decreases the permeability of the surface soil layer, which reduces infiltration rates and the potential for storage of rainfall in the soil matrix. In urban areas, increased impervious cover also reduces the potential for infiltration and soil storage of rainwater.

The reduction of natural storage (i.e., interception, depression, and soil storage) causes changes in runoff characteristics. Specifically, both the total volume and the peak of the surface (or direct) storm runoff increase. The loss of natural storage also causes changes in the timing of runoff, specifically a decrease in both the time to peak and the time of concentration. Runoff velocities are increased, which can increase surface rill and gully erosion rates. Higher stream velocities may also increase rates of bed-load movement.

Land development is often accompanied by changes to drainage patterns and channel characteristics. For example, channels may be cleared of vegetation and straightened, with some also being lined with concrete or riprap. Modifications to the channel may result in decreases in channel storage and roughness, both of which can increase flow velocities and the potential for flooding at locations downstream from the developing area.

Recognizing the potential effects of these changes in runoff characteristics on the inhabitants of the local community, various measures have been proposed to offset these reductions in natural storage. The intent of stormwater management (SWM) is to mitigate the hydrologic impacts of this lost natural storage, usually using manmade storage. Although a variety of SWM alternatives have been proposed, the stormwater management basin remains the most popular. The SWM basin is frequently referred to as a detention or retention basin, depending on its effects on the inflow hydrograph. For our purpose here, the terms will be used interchangeably because the fundamental hydrologic concepts behind each are the same.

To mitigate the detrimental effects of land development, SWM policies have been adopted with the intent of limiting peak flow rates from developed areas to those that occurred prior to development. In addition to specifying the conditions under which SWM methods must be used, these policies indicate the intent of SWM. Specifically, the intent of many SWM policies is to limit runoff characteristics after development to those that existed prior to development. This intent can be interpreted to mean that the flood frequency curve for the post-development conditions coincides with the curve for the pre-development conditions. However, policy statements usually specify one or two exceedence frequencies (i.e., return periods) at which the post-development peak rate must not exceed the pre-development peak rate for the same exceedence frequency. Such policies often use return periods of 2, 10, or 100 years as the target points on the frequency curve.

Where channel erosion is of primary concern, a smaller return period, such as the 6-month event, may serve as the target event. Policies may also specify a specific design method to be used in the design of a SWM control method. Although data do not exist to show that any one method is best, the designation of a specific method as part of a SWM policy will ensure design consistency.

8.1 CLASSIFICATION

Figure 8.1 shows a schematic of the cross-section of a detention basin with a single-stage riser. A pool is formed behind the retaining structure. The hydrograph of the post-development flood runoff enters the pool at the upper end of the detention basin. Water can be discharged from the pool through a pipe that passes through or around the detention structure. The size of the pipe can serve to limit the outflow rate, thus forming a permanent pool, with the permanent-pool elevation changing only through evaporation and infiltration losses.

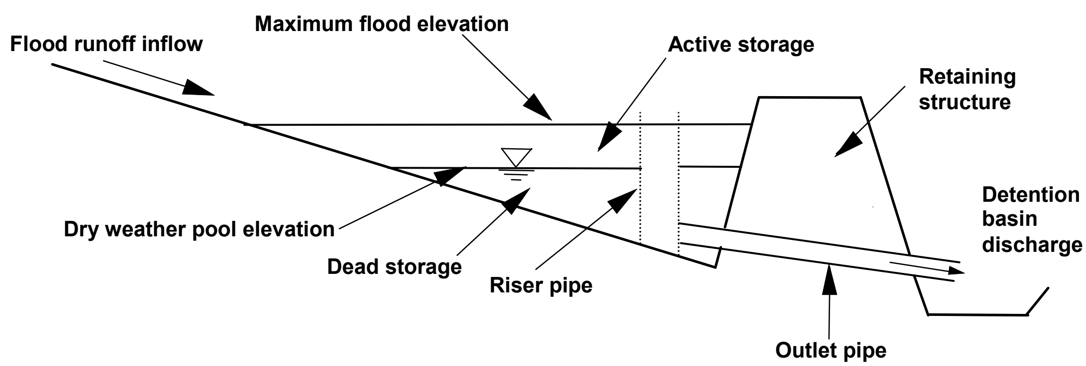


Figure 8.1. Schematic cross-section of a detention basin with a single-stage riser

The use of a permanent pool has a number of advantages, including water quality control, aesthetic considerations, and wildlife habitat improvement. Of course, a permanent pool also increases the total storage volume, which requires both a larger retaining structure and a larger commitment of land, both of which increase the cost of the project.

Figure 8.1 does not show several other elements of detention basin design. The riser inlet may need to be fitted with both an antivortex device and a trash rack. The antivortex device prevents the formation of a vortex, thus maintaining the hydraulic efficiency of the outlet structure. The trash rack prevents trash (and people) from being sucked into the riser by high-velocity flows. Antiseep collars can be fitted to the outside of the discharge pipe to prevent erosion about the pipe within the retaining structure if the seepage gradient exceeds the critical gradient.

All detention basins should have an emergency spillway to pass runoff from very large flood events, so the retaining structure is not overtopped and washed out. The elevation of the bottom of the emergency spillway, which will pass high flows around the retaining structure, is above the elevation of the riser outlet, but below the top of the retaining structure. The zone between is called the detention or surcharge storage zone.

A number of methods have been proposed for use in the planning and design of stormwater detention facilities. Design requires the simultaneous sizing of both the storage volume characteristics and the riser/outlet characteristics. Some SWM methods can only be used to estimate the volume of storage that would be required to meet the intent of the SWM policies. Such methods will be referred to herein as planning methods. Other planning methods are used to determine the characteristics of the outlet facility. Ultimately, the final design should be determined using a method that simultaneously estimates the volume of storage and the characteristics of the outlet facility.

The simultaneous solution is important because there are a wide array of feasible solutions for any one site and set of design conditions. The separate determination of the volume of storage and the characteristics of the outlet facility can lead to an ineffective, and possibly incorrect, design. In summary, planning methods are less accurate and require less effort than design methods.

8.1.1 Analysis versus Synthesis

The problem of analysis versus synthesis is best evaluated in terms of systems theory. The problem is viewed in terms of the input (inflow runoff hydrograph), output (outflow hydrograph), and the transfer function (stage-storage-discharge relationship). In the analysis phase, the two hydrographs would be measured for an existing stormwater management facility, and it would be necessary to calibrate the stage-storage-discharge relationship. While the stage-storage relationship could be determined from topography, the stage-discharge relationship would have to be analyzed. For a given storage facility, the physical characteristics of the outlet facility would be known. Therefore, the analysis would involve determining the best values of the weir and/or orifice coefficients for the outlet. Given the cost involved in data collection, analyses are rarely undertaken; therefore, only the synthesis case will be discussed in this manual.

In the synthesis case, the objective is to make estimates of either the outflow hydrograph or the necessary characteristics of the proposed riser. For watershed studies where detention basins exist, it may be necessary to synthesize flood hydrographs for the detention basin outflow. In this case, the outflow hydrograph is estimated from a design storm. The standard procedure is to assume a design storm and a unit hydrograph. Rainfall excess is computed from the design storm and then the rainfall excess is convolved with the unit hydrograph. The resulting direct runoff hydrograph is used as the design input (inflow runoff hydrograph). Weir coefficients are assumed along with the linear storage equation of Chapter 7 to compute the outflow hydrograph of direct runoff.

The second case of synthesis, which will be referred to as the problem of design, has the objective of estimating the characteristics of the riser/outlet facility in order to meet some design objective. In this case, the output of the design problem is the area of the orifice or the weir length, riser and conduit diameters, and outlet facility elevation characteristics. Unlike the analysis case, the weir or orifice coefficient is assumed, as is the design criterion. This is unlike the watershed evaluation case outlined in the previous paragraph.

8.1.2 Planning versus Design

A number of detention basin planning methods have been proposed in the professional literature. These provide estimates of the required volume of detention storage. The outlet structure is sized independently of the detention volume determination. These methods will be classed as planning methods, although they are occasionally used for design. They are referred to as planning methods because the riser characteristics and volume are determined independently of each other.

Design techniques differ in two ways from the planning methods. First, the planning methods only require peak discharge estimates, as opposed to requiring entire flood hydrographs. Thus routing hydrographs through the detention basin is not necessary when using these planning methods. Second, since routing is not required, a stage-storage-discharge relationship is not required; instead, a "standard" storage-discharge relationship is inherent in the planning methods. A design method uses flood hydrographs, routing, and a site-specific stage-storage-discharge relationship. For this reason, a design method will be more accurate than the planning method. However, the planning methods are much easier to apply. Hence the terms planning and design are used to distinguish between approaches to SWM problem solving that reflect differences in expected accuracy, as well as the cost and effort involved.

The problem of planning the detention facility is separated into two parts, estimating the volume of storage and sizing the characteristics of the outlet facility.

8.2 ESTIMATING DETENTION VOLUMES

A number of methods have been proposed and are being used for estimating detention volumes. Recognizing that these methods often yield widely different estimates, a brief comparison of some of the more widely used methods is in order. A relationship between the ratio of the storage volume to the runoff volume and the ratio of the "pre-development" and "post-development" peak discharges is the basis for many of these methods. For SWM policies that require the peak discharge out of the SWM basin to be no greater than the pre-development peak discharge, the before-to-after ratio is often referred to as the ratio of the outflow to inflow since the peak of the outflow from the detention basin equals the pre-development peak discharge and the inflow to the detention basin equals the post-development peak discharge.

8.2.1 The Loss-of-Natural-Storage Method

The loss-of-natural-storage method for estimating detention volumes is based on the idea that the volume of manmade storage (Q_s) equals the volume of lost natural storage:

$$Q_s = Q_a - Q_b \tag{8.1}$$

in which Q_a and Q_b are the depths (mm or in) of runoff for the post-development and pre-development watershed conditions. It is important to note that the variable Q is often referred to as a volume even though it has the dimension of a depth. While it is actually a depth, when it is referred to as a volume, the assumption is made that it is an equivalent depth spread uniformly over the entire watershed. The volume of storage, V_s, in m^3 (ft^3), is computed by multiplying Q_s by the drainage area A in hectares (acres):

$$V_S = \alpha A Q_S \tag{8.2}$$

where α is a conversion constant equal to 10 in SI and 3,630 in CU units.

The runoff depths Q_a and Q_b of Equation 8.1 can be computed using any one of a number of methods. For the SCS method, the SCS runoff equation (Equation 5.19) can be used with the post-development and pre-development curve numbers (CNs). If the rational method is used to estimate peak discharges, runoff depths Q can be estimated using the peak discharge q_p in m^3/s (ft^3/s), the time of concentration t_c in minutes, and the drainage area A in ha (ac):

$$Q = \alpha \left(\frac{q_p t_c}{A} \right) \tag{8.3}$$

where α is a conversion constant equal to 6 in SI and 1/60.5 in CU units.

Equation 8.3 can be solved for both the pre- and post-development conditions using the appropriate values of q_p and t_c. Then the values are entered into Equation 8.1 to compute the depth of storage, which is then used to compute the volume of storage with Equation 8.2.

Example 8.1. A 2.3 ha (5.7 ac) watershed is being developed. Existing conditions have a rational coefficient C of 0.2 and a time of concentration of 18 minutes. In the developed condition, the coefficient C will be 0.45, and the time of concentration will be 11 minutes. Using the local IDF curve, the rainfall intensities for the existing and developed conditions are 79 mm/h (3.1 in/h) and 102 mm/h (4.0 in/h), respectively.

The peak discharges for the existing and developed conditions are:

CIA

	Value in SI	Value in CU
$q_{pb} = \frac{1}{\alpha} C_b i_b A$	$= \frac{(0.2)(79)(2.3)}{360} = 0.10\ m^3/s$	$= \frac{(0.2)(3.1)(5.7)}{1} = 3.5\ ft^3/s$
$q_{pa} = \frac{1}{\alpha} C_a i_a A$	$= \frac{(0.45)(102)(2.3)}{360} = 0.29\ m^3/s$	$= \frac{(0.45)(4.0)(5.7)}{1} = 10.3 ft^3/s$

Thus, the depths of runoff are computed with Equation 8.3:

	Value in SI	Value in CU
$Q_b = \alpha \left(\frac{q_{pb} t_c}{A} \right)$	$= 6 \left(\frac{0.10(18)}{2.3} \right) = 4.7\ mm$	$= \left(\frac{1}{60.5} \right) \left(\frac{3.5(18)}{5.7} \right) = 0.18\ in$
$Q_a = \alpha \left(\frac{q_{pa} t_c}{A} \right)$	$= 6 \left(\frac{0.29(11)}{2.3} \right) = 8.3\ mm$	$= \left(\frac{1}{60.5} \right) \left(\frac{10.3(11)}{5.7} \right) = 0.33\ in$

The depth of storage (Equation 8.1) and volume of storage (Equation 8.2) are:

	Value in SI	Value in CU
$Q_s = Q_a - Q_b$	$= 8.3 - 4.7 = 3.6$ mm	$=0.33 - 0.18 = 0.15$ in
$V_s = \alpha(A)(Q_s)$	$=10(2.3)(36) = 83\ m^3$	$=3630(5.7)(0.15) = 3100\ ft^3$

8.2.2 The Rational Formula Hydrograph Method

Given the popularity of the rational method, a number of detention volume estimation methods have been developed using the rational method. These methods typically assume a triangular-shaped hydrograph with a time base equal to $2t_c$. One method uses the difference between the post-development and pre-development peak discharges and the post-development time of concentration t_{ca}:

$$V_s = 60\ (q_{pa} - q_{pb})\ t_{ca} \qquad (8.4)$$

where,

V_s = storage volume, m^3 (ft^3)

t_{ca} = post-development time of concentration, min

q_{pa} and q_{pb} = post- and pre-development peak discharges, m^3/s (ft^3/s).

The relationship between these parameters is shown in Figure 8.2. Both q_{pa} and q_{pb} are computed with the rational formula of Equation 5.12.

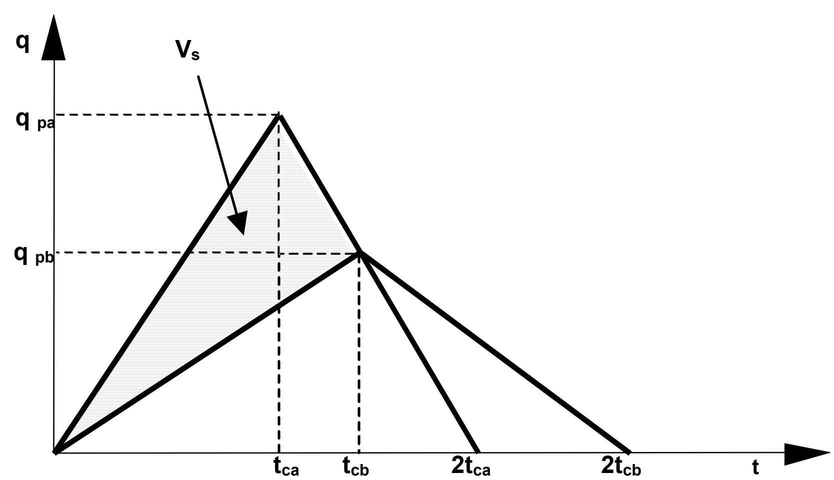


Figure 8.2. Volume of storage (V_s) determination for the rational formula hydrograph method

Example 8.2. Because of development within a 6-ha (14.8-ac) watershed, a detention basin is planned upstream of an existing roadway to prevent ponding at the culvert. Pre- and post-development peak discharges of 0.34 and 0.83 m^3/s (12 and 29 ft^3/s) were computed with the rational method. The post-development time of concentration is 13 minutes. Thus, the required volume of storage is:

	Value in SI	Value in CU
V_s	$= 60(0.83 - 0.34)(13) = 382\ m^3$	$= 60(29 - 12)(13) = 13{,}300\ ft^3$

At an average depth of 1.4 m (4.6 ft), the pond will have an average area of 273 m^2 (2,890 ft^2).

8.2.3 The SCS TR-55 Method

Chapter 6 of SCS Technical Release 55, or TR-55 (SCS, 1986), provides a method for quickly analyzing effects of a storage reservoir on peak discharges. It is based on average storage and routing effects for many structures that were evaluated using the TR-20 method (SCS, 1984). The ratio of the depth of storage to the depth of runoff (Q_s/Q_a) is given as a function of the ratio of the peak rate of outflow to the peak rate of inflow (R_q). The relationship between Q_s/Q_a and R_q is:

$$R_s = \frac{Q_s}{Q_a} = C_o + C_1 R_q + C_2 R_q^{\ 2} + C_3 R_q^{\ 3} \qquad (8.5)$$

in which C_O, C_l, C_2, and C_3 are coefficients (see Table 8.1) that are a function of the SCS rainfall distribution. The volume of storage (m^3 or ft^3) is computed by:

$$V_s = \alpha\, R_s\, Q_a\, A \qquad (8.6)$$

where,

α = conversion constant equal to 10 in SI and 3,630 in CU units
Q_a = post-development depth of runoff, mm (in)
A = drainage area, ha (ac).

Table 8.1. Coefficients for the SCS Detention Volume Method

Rainfall Distribution	C_o	C_1	C_2	C_3
I or IA	0.660	-1.76	1.96	-0.730
II or III	0.682	-1.43	1.64	-0.804

Example 8.3. Development within a 7.3-ha (18-ac) watershed is planned near a local roadway. A planning estimate of the storage required to detain runoff from a 100-mm (3.9-in) storm is needed. The curve number for existing conditions is 70, and development within the watershed will increase the CN to 80. The pre- and post-development times of concentration are 0.55 hour and 0.37 hour, respectively.

The pre- and post-development runoff depths are obtained from the SCS runoff equation with values of 33 mm (1.3 in) and 51 mm (2.0 in), respectively. The I_a/P ratios are 0.22 and 0.13. From Equation 5.21, the unit peak discharges are 0.194 $m^3/s/km^2/mm$ and 0.238 $m^3/s/km^2/mm$ (449 $ft^3/s/mi^2/in$ and 553 $ft^3/s/mi^2/in$), assuming a type II rainfall distribution. Thus, the pre-development and post-development peak discharges are:

	Value in SI	Value in CU
q_{pb}	= 0.194 (0.073 km^2) (33 mm) = 0.467 m^3/s	= 449 (18/640 mi^2) (1.3 in) = 16.4 ft^3/s
q_{pa}	= 0.238 (0.073 km^2) (51 mm) = 0.886 m^3/s	= 553 (18/640 mi^2) (2.0 in) = 31.1 ft^3/s

These values yield a discharge ratio R_q of 0.527, which is used as input to Equation 8.5 to obtain the volume ratio R_s:

$$R_s = 0.682 - 1.43(0.527) + 1.64(0.527)^2 - 0.804(0.527)^3$$
$$= 0.27$$

Thus, the volume of storage is computed using Equation 8.6:

	SI Unit	CU Unit
$V_s = \alpha R_s Q_a A$	$= 10(0.27)(51)(7.3) = 1000\ m^3$	$= 3630(0.27)(2.0)(18) = 35{,}300\ ft^3$

8.2.4 Actual Inflow/Estimated Release

The actual inflow/estimated release method requires an inflow hydrograph and a target maximum release value. An estimated release rate from the storage facility is drawn on a graph of the inflow hydrograph as shown in Figure 8.3. The release rate is usually assumed to be a

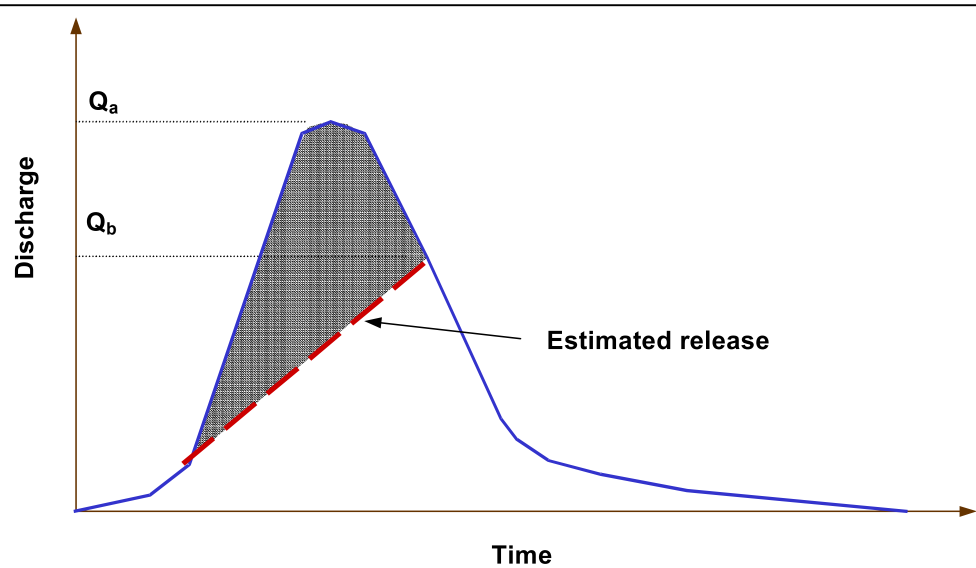


Figure 8.3. Storage volume estimate using actual inflow/estimated release

straight line from near the beginning of the rising limb of the inflow hydrograph to the point on the receding limb of inflow hydrograph equal to the maximum release rate. The required volume is represented by the area between the inflow hydrograph and the estimated release rate.

8.3 WEIR AND ORIFICE EQUATIONS

Weirs and orifices are engineered devices that can be used to control and measure flow rates. While these devices can occur naturally, for the context of engineering design, the discussion will center on the equations used in the design of hydrologic/hydraulic facilities.

8.3.1 Orifice Equation

Figure 8.4 shows a schematic of a tank with a hole of area A_2 in its bottom. Assuming all losses can be neglected, Bernoulli's equation can be written between a point on the surface of the pool (point 1) and a point in the cross-section of the orifice (point 2):

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + z_2 \qquad (8.7)$$

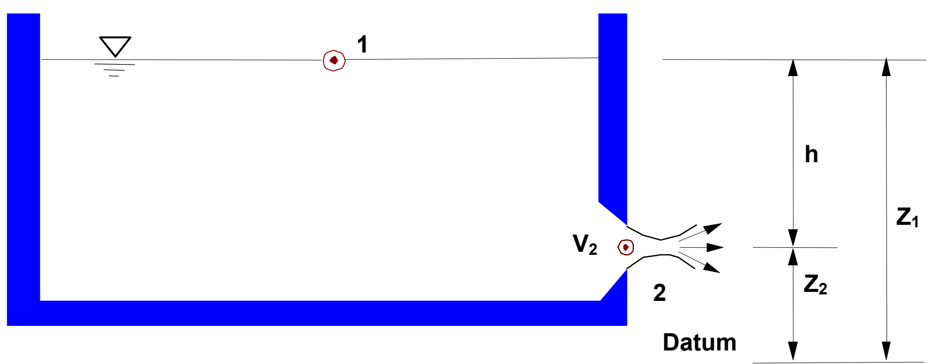


Figure 8.4. Schematic diagram of the flow through an orifice

This can be simplified by making the following assumptions: (1) the pressure at both points is atmospheric, therefore $p_1 = p_2$; (2) the surface area of the pool A_1 is very large relative to the area of the orifice A_2, so from the continuity equation V_1 is essentially zero; and (3) $z_1 - z_2 = h$. Thus, Equation 8.7 becomes

$$h = V_2^2/2g \qquad (8.8)$$

Solving for V_2 and substituting it into the continuity equation yields:

$$Q = A\ V = A\sqrt{2\,g\,h} \qquad (8.9)$$

Equation 8.9 depends on two assumptions that are not always true: zero losses and atmospheric pressure across the opening of the orifice. It is actually atmospheric at a point below the orifice, where the cross-sectional area of the discharging water is slightly smaller than the area of the orifice. Because of these assumptions, the discharge will be less than that given by Equation 8.9. The actual discharge through the orifice is estimated by applying a discharge coefficient C_d to Equation 8.9:

$$Q = C_d A \sqrt{2 g h} \tag{8.10}$$

in which C_d is called the discharge coefficient and is dimensionless. For some design problems, the Q of Equation 8.10 is multiplied by an efficiency factor f to reflect other types of losses that limit the discharge rate. Values of C_d range from 0.5 to 1.0, with a value of 0.6 commonly used. If the orifice is not horizontal, the depth h is usually measured from the center of area of the orifice.

If the opening of the orifice is partially or fully submerged on the downstream side, the discharge through the orifice is reduced. Equation 8.10 remains applicable except that h is defined as the difference between the water surface elevations upstream and downstream of the orifice.

Example 8.4. For a particular detention basin being planned, the peak discharge must be limited to 0.55 m^3/s (19.4 ft^3/s). At maximum stage for the design storm, the water depth above the center of area of the orifice is 0.7 m (2.3 ft). The need is to estimate the area of the orifice in the riser pipe that will be used to limit the discharge to the allowable rate. Assuming a discharge coefficient of 0.6, Equation 8.13 can be used to solve for the area of the orifice:

	Value in SI	Value in CU
$A = \frac{q}{C_d \sqrt{2gh}}$	$= \frac{0.55}{0.6\sqrt{2(9.81)(0.7)}} = 0.247 \; m^2$	$= \frac{19.4}{0.6\sqrt{2(32.2)(2.3)}} = 2.66 \, ft^2$

Thus, a 0.247 m by 1 m (1 ft by 2.66 ft) orifice, or many other possible configurations, would limit the discharge to 0.55 m^3/s (19.4 ft^3/s).

8.3.2 Weir Equation

Consider the cross-section shown in Figure 8.5. Point 1 is located at a point upstream of the obstruction at a distance where the obstruction does not influence the flow characteristics. Point 2 is at the obstruction. The following analysis assumes: (1) ideal flow, (2) frictionless flow, (3) critical flow conditions at the obstruction, and (4) the obstruction has a unit width perpendicular to the direction of flow.

For the critical flow conditions, the following equations describe hydraulic conditions at the obstruction:

$$F_r = 1 = \frac{V_c}{(g d_c)^{0.5}} \tag{8.11}$$

$$d_c = \left(\frac{q_u^2}{g} \right)^{1/3} \tag{8.12}$$

$$E_c = d_c + \frac{V_c^2}{2g} = \frac{3}{2} d_c \qquad (8.13)$$

where,

F_r = Froude number
V_c = critical velocity
d_c = critical depth
q_u = discharge rate per unit width
E_c is the minimum specific energy.

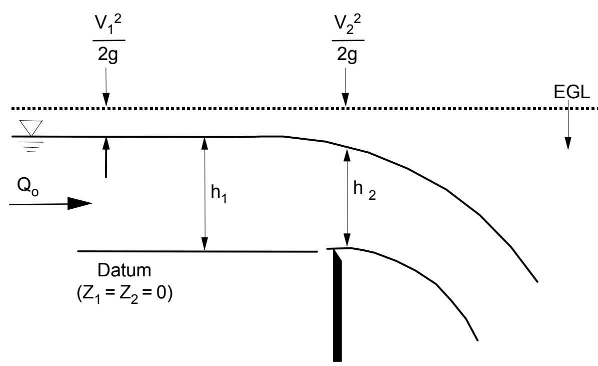


Figure 8.5. Schematic diagram of flow over a sharp-crested weir

If hydrostatic pressure is assumed at sections 1 and 2, then $P_i/\gamma = h_i$. Thus, Bernoulli's equation is:

$$h_1 + \frac{V_1^2}{2g} + z_1 = h_2 + \frac{V_2^2}{2g} + z_2 \qquad (8.14)$$

By setting the datum at the top of the weir, $z_1 = z_2 = 0$, assuming that the velocity head at section 1 is much smaller than the velocity head at section 2, and recalling that the flow passes through critical depth as it passes over the weir, then $V_2 = V_C$ and $h_2 = d_C$, Equation 8.14 reduces to:

$$h_1 = \frac{V_c^2}{2g} + d_c \qquad (8.15)$$

From Equation 8.11, the velocity head is $V_c^2/2g = d_c/2$. Defining $h = h_1$, then:

$$h = \frac{d_c}{2} + d_c = \frac{3\,d_c}{2} \quad or \quad d_c = \frac{2}{3} h \qquad (8.16)$$

Solving Equation 8.15 for q_u, it then follows that:

$$q_u = (g\,d_c^3)^{0.5} = \left[g\left(\frac{2}{3}h\right)^3\right]^{0.5} = \left(\frac{8g}{27}\right)^{0.5} h^{3/2} = \left(\frac{2}{\sqrt{27}}\right)\sqrt{2g}\,h^{3/2} \qquad (8.17)$$

Letting Q = q_uL, the general weir equation is:

$$Q = \left(\frac{2}{\sqrt{27}}\right)\sqrt{2g}\,h^{3/2}L \qquad (8.18)$$

where,

Q = discharge over a horizontal weir, m^3/s (ft^3/s)
h = head (depth) of approach flow above the weir, m (ft)
L = weir length, m (ft)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2).

Equation 8.18 represents ideal flow over a weir. Actual weirs will perform less efficiently, therefore, the equation is modified by the addition of a weir coefficient, C_w, whose value is dependent on the type of weir, head, weir height and other factors and includes the initial quotient in Equation 8.18. It is significant to note that many presentations of the weir equation embed the $\sqrt{2g}$ into the coefficient, C_w , making it a dimensioned rather than dimensionless quantity. By keeping the gravity term in the equation, C_w becomes a property of a specific weir type and not dependent on the system of units.

$$Q = C_W\,\sqrt{2g}\,h^{3/2}L \qquad (8.19)$$

where,

C_w= dimensionless weir coefficient

For the sharp-crested weir of this derivation, values of C_w can range from 0.27 to 0.38. The range reflects the variation in losses from alternative weir/flow configurations. Losses depend on the depth of flow over and approaching the weir, weir length, weir thickness, and weir height. Accurate estimates of C_w are difficult to obtain even in laboratory studies. Generally, C_w increases with increasing flow depth and decreases with increases in either weir length or weir height. A value of 0.37 can be used for sharp-crested rectangular weirs where more information is not available.

Weir coefficient also varies with the type of weir (sharp-crested or broad-crested) and the shape of the weir (triangular, rectangular, etc.). Brater, et al. (1996) and other references report weir coefficients for a variety of weir types and conditions. When consulting other sources for weir coefficients, it should be noted that some references report weir coefficient as a dimensional quantity that differs depending on the applicable unit system. In this formulation, the weir coefficient is dimensionless.

If the downstream face of a weir is submerged, the discharge passing over the weir is reduced. The discharge under submerged conditions, Q_s, is computed as a function of the unsubmerged discharge, Q, the head, h, and the downstream head, h_s:

$$Q_s = Q\left[1 - \left(\frac{h_s}{h}\right)^{3/2}\right]^{0.385} \qquad (8.20)$$

Example 8.5. An existing detention basin near the site of a project where highway drainage is being renovated has a weir length of 2 m (6.6 ft) and a weir coefficient of 0.37. The pond was sized such that the depth of ponded storm water at flood stage is 1.1 m (3.6 ft). The discharge passing the weir is needed to assess the adequacy of highway drainage. The discharge is:

	Value in SI	Value in CU
$Q = C_W\sqrt{2g}\ h^{3/2}L$	$= 0.37\sqrt{2(9.81)}\ (1.1)^{1.5}\ (2)$ $= 3.6\ m^3/s$	$=0.37\sqrt{2(32.2)}\ (3.6)^{1.5}(6.6)$ $= 134\ ft^3/s$

8.4 SIZING OF DETENTION BASIN OUTLET STRUCTURES

The methods described in Section 8.2 can be used only to estimate the volume of detention storage. The second step necessary to size a detention basin is the determination of the physical characteristics of the outlet structure. The outlet may be based on either a weir or an orifice, or both. Hydraulic procedures such as those given by Normann, et al. (1985) can be applied. The following hydrologic procedures are commonly used for small structures.

Figure 8.1 shows a schematic of a basin with a pipe outlet. In addition to determining the diameter of the pipe barrel for a pipe outlet facility, it is also necessary to establish elevations of the pipe inlet and outlet. For those policies that require a permanent pool (i.e., wet pond), both the volume of dead storage and the corresponding elevation of the permanent pool must be set.

Both the size and effectiveness of a detention basin are largely dependent on the exceedence frequency (i.e., return period). Studies have shown that a basin designed for single-stage control of a frequent event (i.e., 2- or 5-year event) will tend to over control the less frequent events (i.e., 50- or 100-year events). Conversely, a basin designed for single-stage control of a less frequent event will tend to under control the more frequent events. An outlet facility sized to pass the 2-year event will not allow the 100-year event to pass with the same speed that a pipe outlet sized for a 100-year event will pass through; thus, over control results.

Initially, most SWM policies required a single-stage riser. More enlightened SWM policies use two-stage control because of the problems of under control and over control associated with single-stage risers. The sizing of both single-stage and two-stage risers will be discussed here.

In the sizing of risers, it is necessary to determine both the required volume of storage and the physical characteristics of the riser. The physical characteristics include the outlet pipe diameter, the riser diameter, either the length of the weir or the area of the orifice, and the elevation characteristics of the riser. Single-stage risers with weir flow and orifice flow are shown in Figures 8.6a and b, respectively. For weir flow control, Equation 8.20 defines the relationship between the discharge Q and (1) the depth, h, above the weir; (2) the discharge or weir coefficient, C_w; and (3) the length of the weir, L_w. Equation 8.10 provided the general formula for flow through an orifice.

8.4.1 Single-Stage Risers

The procedure given in this section provides guidance on sizing a single-stage riser. Single-stage risers, as shown in Figure 8.6, provide control of runoff for cases where practice specifies one exceedence frequency. A procedure for two-stage risers is given in Section 8.6.

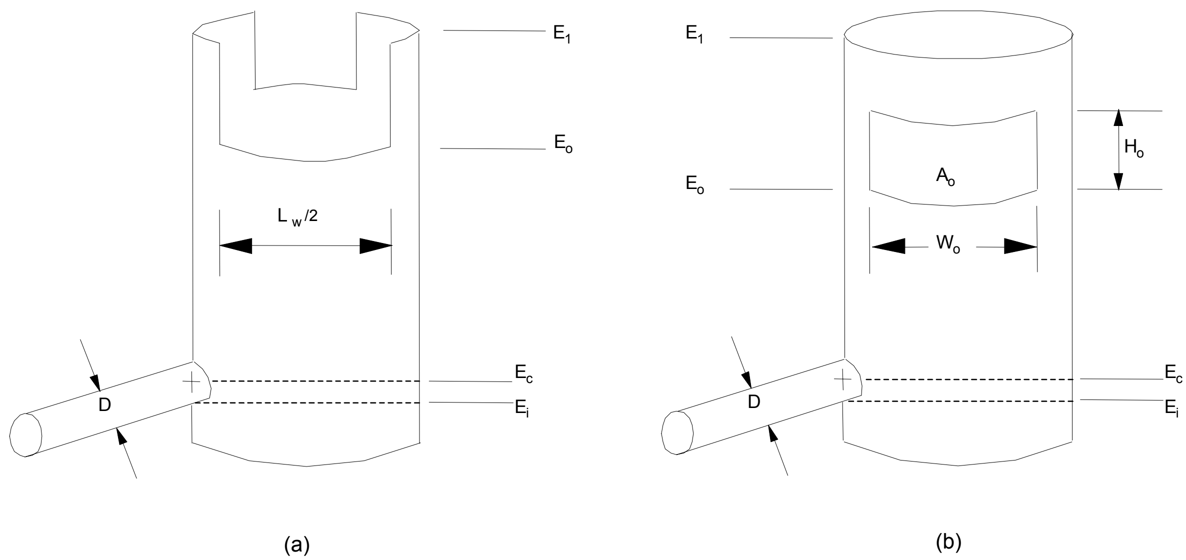


Figure 8.6. Single-stage riser characteristics for (a) weir flow and (b) orifice (or port) flow

8.4.1.1 Input Requirements and Output

Estimating the characteristics of a riser requires the following inputs: (1) watershed characteristics, including area, pre- and post-development times of concentration, and pre- and post-development curve numbers (assuming SCS CN procedures are used for abstractions); (2) rainfall depth(s) for the design storm(s); (3) characteristics of the riser and outlet pipe structure, including pipe roughness (n), length, and an initial estimate of the diameter; (4) elevation information, including stage vs. storage values, the wet-pond elevation, if applicable, and the elevation of the centerline of the pipe; and (5) hydrologic and hydraulic models, including a model for estimating peak discharges and runoff depths, a model for estimating the volume of storage as a function of pre- and post-development peak discharges, and a model for estimating weir and orifice coefficients, as necessary.

The output from the analysis includes the following: (1) the length of the weir or the area of the orifice; (2) the depth and volume of storage; (3) elevations of riser characteristics; and (4) the diameter of the outlet pipe.

8.4.1.2 Procedure for Sizing the Riser

In the following procedure, both the riser characteristics and the volume of storage will be estimated. The following steps (adapted from Woodward, 1983) are used to size a single-stage riser:

1. Design the culvert barrel using the procedures of HDS-5 (Normann, et al., 1985) and set the culvert invert elevation. It is preferable to design the culvert so that the maximum headwater under the maximum design discharge from the outlet structure is less than or equal to the invert of the orifice opening (or weir). If this is not the case, the opening must be designed accounting for submerged flow.

2. Estimate the volume of storage, V_s, required (Section 8.2).

3. Using the elevation E_o of either the weir or the bottom of the orifice, obtain the volume of dead storage V_d from the elevation-storage curve.

4. Compute the total storage:

$$V_t = V_d + V_s \tag{8.21}$$

5. Enter the elevation-storage curve with V_t to obtain the water surface elevation, E_l.

6. Size the opening. If an orifice go to (a), if a weir go to (b).

   a. If the outlet is an orifice, determine the characteristics of the orifice:

      (i) Assume an orifice height, H_o.

      (ii) Compute the area of the orifice opening assuming unsubmerged flow, A:

$$A_o = \left(\frac{1}{C_d\sqrt{2g}}\right)\frac{Q_{pb}}{\left(E_1 - E_0 - \frac{H_o}{2}\right)^{0.5}} \tag{8.22}$$

      where,

      A_o = orifice opening area, m^2 (ft^2)
      Q_{pb} = discharge through the orifice (pre-development), m^3/s (ft^3/s)
      E_1 = water surface elevation upstream of the orifice, m (ft)
      E_o = elevation of the bottom of the orifice, m (ft)
      g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2).

      H_o/2 adjusts for head being measured from the center of the orifice.

      (iii) For a rectangular orifice, compute the width of the orifice opening W_o:

$$W_o = \frac{A_o}{H_o} \tag{8.23}$$

b. If the outlet is a weir, determine the weir length for unsubmerged flow:

$$L_w = \left(\frac{1}{\sqrt{2g}\, C_w} \right) \frac{Q_{pb}}{\left(E_1 - E_0\right)^{1.5}} \tag{8.24}$$

7. Verify the design performance using storage routing. (The approximate procedures for estimating storage volume do not account for performance of the outlet structure throughout the passage of the inflow hydrograph and may result in an under- or over-design.)

Example 8.6. A single stage riser must be designed for a pond serving a 4.53 ha (11.2 ac) watershed. It must reduce post-development 2-year peak of 0.5 m^3/s (17.7 ft^3/s) to the pre-development peak of 0.08 m^3/s (2.8 ft^3/s). The outlet structure will consist of an outlet pipe culvert and a riser with a rectangular orifice. The bottom of the pond will be located at an elevation of 29.5 m (96.8 ft). The pond will include permanent pool elevation at 30.1 m (98.8 ft) (dead storage) for water quality purposes.

First, a culvert barrel is selected and located so that the headwater at the culvert entrance under the design discharge of 0.08 m^3/s (2.8 ft^3/s) does not reach the invert of the orifice and force it to operate under submerged conditions. Using the procedures of HDS-5, and considering the site constraints on culvert slope and invert location, a 400 mm (16 in) diameter, 10.5 m (34.4 ft) long, reinforced concrete pipe (n=0.012) was selected. (If the selected size is not available, the next larger available size should be specified.) The entrance invert is to be placed at an elevation of 29.3 m (96.1 ft).

Details on constructing a stage-storage relationship are given in Section 8.5. For this example, the stage-storage relationship at the site of the detention structure is given below. Based on the permanent pool elevation, the dead storage can be interpolated from the stage-storage curve (see Table 8.2) to be 380 m^3 (13,400 ft^3).

The active storage is computed using one of the methods presented in Section 8.2 and is estimated at 670 m^3 (23,700 ft^3). The total storage is the sum of the active and dead storages, which is 1050 m^3 (37,100 ft^3).

The elevation corresponding to the total storage is found by interpolation of the stage-storage curve, which is 30.9 m (101.4 ft)

Table 8.2. Stage-Storage Curve

SI		CU	
Elevation (m)	**Volume (m^3)**	**Elevation (ft)**	**Volume (ft^3)**
29.5	0	96.8	0
29.6	60	97.0	1390
29.8	170	97.5	4410
30.0	300	98.0	7620
30.2	460	98.5	11240
30.4	620	99.0	15540
30.6	800	99.5	19850
30.8	980	100.0	24440
31.0	1200	100.5	29280
31.2	1440	101.0	34120
31.4	1710	101.5	39940
31.6	2020	102.0	46170
31.8	2400	102.5	52860
32.0	2760	103.0	60120
		103.5	68420
		104.0	77990
		104.5	88030

The orifice invert was established at an elevation of 30.1 m (98.8 ft) to create the permanent pool. Assuming an initial orifice height of 0.15 m (0.5 ft), the area of the orifice in the riser is computed with Equation 8.22:

	SI	CU
A_0	$= \left(\frac{1}{0.6\sqrt{2(9.81)}}\right)\frac{0.08}{(30.9 - 30.1 - (0.15/2))^{0.5}}$ $= 0.0354\ m^2$	$= \left(\frac{1}{0.6\sqrt{2(32.2)}}\right)\frac{2.8}{(101.4 - 98.8 - (0.5/2))^{0.5}}$ $= 0.379\ ft^2$
W_0	$= \frac{0.0354}{0.15} = 0.24\ m$	$= \frac{0.379}{0.5} = 0.76\ ft$

Thus, one possible orifice is 0.15 m by 0.24 m (0.5 ft by 0.76 ft). If the calculated dimensions are not practical construction sizes, an iterative trial process with practical dimensions resulting in the same performance must be conducted until suitable dimensions are found. It is not usually appropriate to select the next larger available dimensions because this will allow excessive discharge through the orifice. The design should be checked using storage routing before finalizing.

The diameter of the riser (if it is also circular) is usually 2 to 3 times the diameter of the outlet culvert, so a 1.2 m (48 in) riser pipe can be used for the riser.

8.4.2 Two-Stage Risers

Where stormwater or drainage policies require control of flow rates of two exceedence frequencies, the two-stage riser is an alternative for control. The structure of a two-stage riser is similar to the single-stage riser except that it includes either two weirs or a weir and an orifice (see Figure 8.7). For the weir/orifice structure, the orifice is used to control the more frequent event, and the larger event is controlled using the weir. The runoff from the smaller and larger events are also referred to as the low-stage and high-stage events, respectively. Values for variables at low and high stages may be followed by a subscript 1 or 2, respectively; for example, Q_{pb2} will indicate the pre-development peak discharge for the high-stage event. Recognizing that the two events will not occur simultaneously, both the low-stage weir or orifice and the high-stage weir are used to control the high-stage event.

8.4.2.1 Input Requirements and Output

The input requirements for sizing a two-stage riser are expanded from those for a single-stage riser. In addition to the single-stage inputs, it is necessary to specify the types of control (i.e., weir/weir or weir/orifice), the rainfall depth for the second stage, and the discharge coefficient for the second stage. In addition to the output for the single-stage analysis, the size characteristics of the second stage of the riser are computed. The procedure also requires the estimated volume of storage for both storm events.

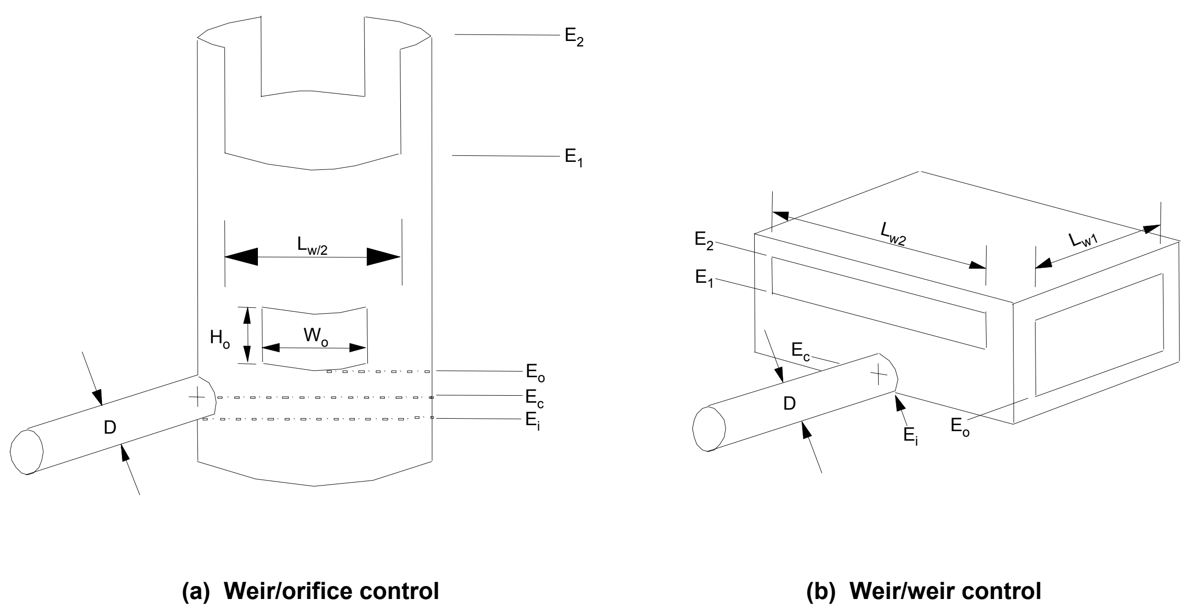


Figure 8.7. Two-stage riser

8.4.2.2 Procedure for Sizing the Riser

The sizing of a two-stage riser expands on the procedure for the sizing of a single-stage riser. The procedure follows the same general format as for the single-stage riser, but both the high-stage weir and low-stage outlet characteristics must be determined. The input for sizing a two-stage riser is the same as that for a single-stage riser, but many of the values must be computed for both the low-stage and high-stage events.

The following steps can be used to size a two-stage riser for the cases where the low-stage outlet is either a weir or an orifice and the high-stage outlet is a weir:

1. Design the culvert barrel using the procedures of HDS-5 (Normann, et al., 1985) and set the culvert invert elevation. It is preferable to design the culvert so that the maximum headwater under the maximum design discharge from the outlet structure is less than or equal to the invert of the orifice opening (or weir). If this is not the case, the opening must be designed accounting for submerged flow. For the two-stage riser, the maximum design discharge corresponds to the high-stage event.

2. Design the low-stage opening as described in Steps 2-6 of Section 8.5.2. If the low-stage opening is a weir of the configuration shown in Figure 8.7(b), it can only be assumed to be operating independently up to an elevation equal to the invert of the high-stage weir.

3. Estimate the high-stage storage volume, V_{s2}, required (Section 8.2).

4. Compute the total high-stage storage:

$$V_{t2} = V_d + V_{s2} \qquad (8.25)$$

5. Enter the elevation-storage curve with V_{t2} to obtain the water surface elevation, E_2.

6. Estimate the discharge through the low-stage opening during the high-stage event. If an orifice go to (a), if a weir go to (b).

    a. If the low-stage outlet is an orifice, the discharge is:

$$Q_{o2} = C_{d1}\sqrt{2g}\ A_{o1}\left(E_2 - E_0 - \frac{H_o}{2}\right)^{0.5} \qquad (8.26)$$

    b. If the low-stage outlet is a weir, the discharge is:

$$Q_{o2} = C_{w1}\ \sqrt{2g}\ L_{w1}(E_2 - E_o)^{1.5} \qquad (8.27)$$

7. Set the invert of the high-stage weir equal to E_1 (maximum elevation during the low-stage event)

8. Compute the high-stage weir length:

$$L_{w2} = \left(\frac{1}{\sqrt{2g}\, C_w} \right) \frac{Q_{pb2} - Q_{o2}}{\left(E_2 - E_1\right)^{1.5}} \quad (8.28)$$

9. Verify the design performance using storage routing. (The approximate procedures for estimating storage volume do not account for performance of the outlet structure throughout the passage of the inflow hydrograph and may result in an under- or over-design.)

Example 8.7. A two-stage riser is required for the 2-year and 100-year events for the location introduced in Example 8.6. (In that example, a one-stage riser was designed to reduce the post-development 2-year peak of 0.5 m^3/s (17.7 ft^3/s) to the pre-development peak of 0.08 m^3/s (2.8 ft^3/s). In this example, we will start with that design, but add the requirement to also reduce the post-development 100-year peak of 1.91 m^3/s (67.4 ft^3/s) to the pre-development 100-year peak of 0.92 m^3/s (32.5 ft^3/s). The outlet structure will consist of an outlet pipe culvert and a riser with a rectangular low-stage orifice and a high-stage weir opening. The bottom of the pond will be located at an elevation of 29.5 m (96.8 ft). The pond will include permanent pool elevation at 30.1 m (98.8 ft) (dead storage) for water quality purposes.

First, the culvert barrel must be re-designed from the earlier example because the maximum design discharge has increased. It must be selected and located so that the headwater at the culvert entrance under the 100-year pre-development design discharge of 0.92 m^3/s (32.5 ft^3/s) does not reach the invert of the orifice and force it to operate under submerged conditions. Using the procedures of HDS-5, and considering the site constraints on culvert slope and invert location, a 900 mm (36 in) diameter, 10.5 m (34.4 ft) long, reinforced concrete pipe (n=0.012) was selected. (If the selected size is not available, the next larger available size should be specified.) The entrance invert is to be placed at an elevation of 29.3 m (96.1 ft).

Details on constructing a stage-storage relationship are given in Section 8.5. For this example, the stage-storage relationship used in Example 8.6 is given used again and is found in Table 8.2. The low-stage (2-year) riser orifice from Example 8.6 is 0.15 m in height by 0.24 m in width (0.5 ft by 0.76 ft).

The active storage for the 100-year event is computed using one of the methods presented in Section 8.2 and is estimated at 1450 m^3 (51,200 ft^3). The total storage is the sum of the active and dead storages, which is 380 + 1450 =1830 m^3 (13,400 + 51,200 = 64,600 ft^3). The elevation corresponding to the total storage is found by interpolation of the stage-storage curve, which is 31.5 m (103.3 ft).

The discharge through the low-stage orifice during the 100-yr event is given by:

	Value in SI	Value in CU
Q_{02}	$= 0.6\sqrt{(2)9.81}(0.036)\left(31.5 - 30.1 - \frac{0.15}{2}\right)^{0.5}$ $= 0.11\ m^3/s$	$= 0.6\sqrt{(2)32.2}\ (0.38)\left(103.3 - 98.8 - \frac{0.5}{2}\right)$ $= 3.8\ ft^3/s$

The invert of the weir should be placed at the maximum elevation in the pond during the low-stage event. From Example 8.6, this was 30.9 m (101.4 ft). The required weir length is then calculated using a weir coefficient, $C_w = 0.37$:

	SI Unit	CU Unit
L_{w2}	$= \left(\frac{1}{\sqrt{(2)9.81}(0.37)}\right)\frac{0.92 - 0.11}{(31.5 - 30.9)^{1.5}}$ $= 1.06\ m$	$= \left(\frac{1}{\sqrt{(2)32.2}(0.37)}\right)\frac{32.5 - 3.8}{(103.3 - 101.4)^{1.5}}$ $= 3.69\ ft$

Thus, the high-stage weir length is 1.06 m (3.69 ft). If the calculated dimension is not a practical construction size, the next smaller available opening should be selected. The design should be checked by storage routing before finalizing.

8.5 DERIVATION OF A STAGE-STORAGE-DISCHARGE RELATIONSHIP

Routing a hydrograph through a reservoir or detention structure requires the relationship between stage, storage, and discharge. The stage-storage-discharge (SSD) relationship is a function of both the topography at the site of the storage structure and the characteristics of the outlet facility. The topographic features of the site control the relationship between stage and storage, and the relationship between stage and discharge is primarily a function of the characteristics of the outlet facility.

8.5.1 The Stage-Storage Relationship

The stage-storage relationship depends on the topography at the site of the storage structure. Consider the unrealistic case of a site where the topography permits a storage structure that has a horizontal rectangular bottom area with vertical sides. In this case, the storage is simply the bottom area (i.e., length times width) multiplied by the depth of storage. If the relationship is plotted in a Cartesian axis system with storage as the ordinate and stage as the abscissa, the stage-storage relationship will be a straight line with a slope equal to the surface area of the storage facility.

For the case where the bottom of the storage facility is a rectangle (L x W), the longitudinal cross- section is a trapezoid with base W and side slopes of angle θ, and the ends are vertical, the stage-storage relationship is given by:

$$S = \frac{L}{\tan\theta} h^2 + (LW)h \tag{8.29}$$

in which h is the height above the bed of the storage facility. If Equation 8.29 is plotted on a graph, the stage-storage relationship has the shape of a second-order polynomial with a zero intercept and a shape that depends on the values of L, W, and θ.

Unless the site undergoes considerable excavation, the simple forms described previously are not "real world." However, the concepts used to derive the stage-storage relationship for the simple forms are also used to derive the stage-storage relationship for an actual site. Instead of a continuous function such as Equation 8.29, the stage-storage relationship is derived as a discrete function (i.e., a set of points). The area within contour lines of the site can be measured, with the storage in any depth increment Δh equal to the product of the average area and the depth increment Δh. Thus, the storage increment ΔS is given by:

$$\Delta S = \frac{1}{2}(A_i + A_{i+1})\Delta h \qquad (8.30)$$

where A_i and A_{i+1} are the surface areas for the i^{th} and $(i + 1)^{th}$ contours.

Example 8.8. Consider the storage facility of Equation 8.29. If the basin has a length of 200 m (656 ft), a width of 100 m (328 ft), and side slopes of 2H:1V, then Equation 8.29 becomes:

$$S = 2Lh^2 + (LW)h$$

and the storage at a depth of 1.5 m (4.9 ft) is calculated as follows:

	Value in SI	Value in CU
S	$= 400\ h^2 + 20{,}000\ h$ $= 400(1.5)^2 + 20{,}000(1.5) = 30{,}900\ m^3$	$= 1{,}312\ h^2 + 215{,}200\ h$ $= 1{,}312(4.9)^2 + 215{,}200(4.9) = 1{,}086{,}000\ ft^3$

Example 8.9. To illustrate the use of Equation 8.30, consider the site shown in Figure 8.8. The area bounded by each contour line was estimated (Table 8.3) and the average area within adjacent contours computed. The topographic lines are drawn with a 1-m contour interval. The stage-storage relationship was computed using Equation 8.30 and is given in Table 8.3 and

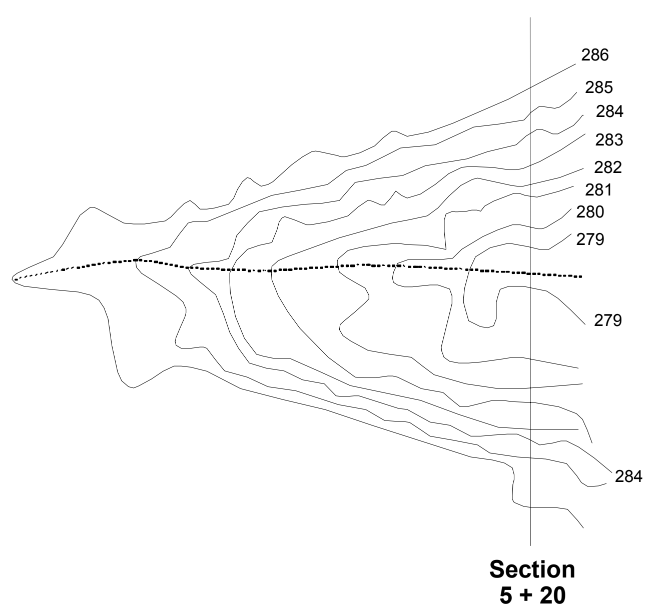


Figure 8.8. Topographic map for deriving stage-storage relationship at site of structure (section 5+20)

Figure 8.9. (All elevations, intervals and depths are in m (ft), areas in m^2 (ft^2), and storage in m^3 (ft^3). This example describes the process for developing a stage-storage curve and does not imply that 1 ft = 1 m.

Table 8.3. Derivation of Stage-Storage Relationship for Example 8.9*

Contour Elevation	Total Area within Contour Elevation	Average Area	Contour Interval	Depth	Change in Storage	Cumulative Storage
278	0			0		0
		245	1		245	
279	490			1		245
		1,115	1		1,115	
280	1,740			2		1,360
		3,015	1		3,015	
281	4,290			3		4,375
		5,585	1		5,585	
282	6,880			4		9,960
		8,600	1		8,600	
283	10,320			5		18,560
		11,575	1		11,575	
284	12,830			6		30,135
		14,165	1		14,165	
285	15,500			7		44,300
		17,685	1		17,685	
286	19,870			8		61,985

* Numerical values are independent of the system of units represented by this example.

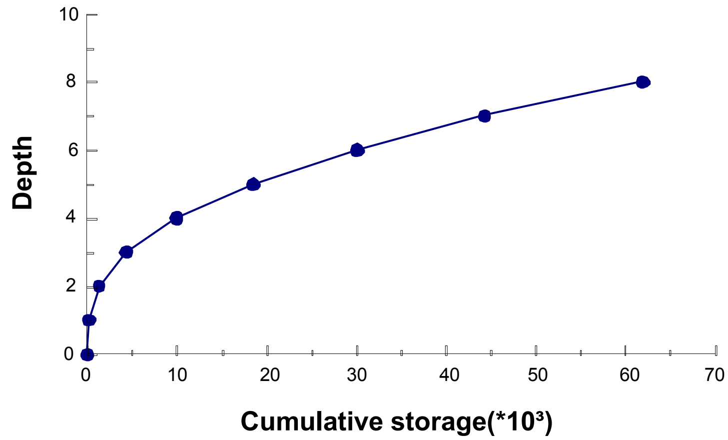


Figure 8.9. Stage-storage relationship

Example 8.10. Many storage facilities are designed to include a permanent pool. In such cases, the elevation of the weir or the bottom of the orifice is set above the elevation of the bottom of the pond. Storage below the elevation of the outlet is called dead storage. Storage above the elevation of the outlet is called active storage. Total storage is the sum of the active and dead storages. (All elevations, intervals and depths are in m (ft), areas in m^2 (ft^2), and storage in m^3 (ft^3). This example describes the process for developing a stage-storage curve and does not imply that 1 ft = 1m.

The data of Table 8.4 can be used to illustrate the development of stage-active storage relationship. Areas are measured from a topographic map at the site of the detention facility. A depth increment of 0.25 is used. The areas are given in column 2 and the average areas are given in column 3. The incremental volumes are the product of the change in depth, 0.25, and the average area. The total storage is the cumulative of the incremental storages.

If the outlet facility has a minimum elevation of 0.5, all storage below this elevation is dead storage. The active storage is 0.0 at an elevation of 0.5. The active storage above 0.5 equals the difference between the total storage and the dead storage. The stage (column 1) versus active storage (column 7) would be used when designing a storage facility.

Table 8.4. Computation of Stage-Active Storage Relationship for Example 8.10

(1) Depth h	(2) Surface Area, A	(3) Average Area, A	(4) Incremental Volume ΔV	(5) Total Volume V	(6) Dead Storage V_d	(7) Active Storage V_a
0	2,000			0	0	0
		2,100	525.0			
0.25	2,200			525.0	525.0	0
		2,250	562.5			
0.50	2,300			1087.5	1087.5	0
		2,400	600.0			
0.75	2,500			1687.5	1087.5	600.0
		2,650	662.5			
1.00	2,800			2350.0	1087.5	1262.5
		2,900	725.0			
1.25	3,000			3075.0	1087.5	1987.5
		3,050	762.5			
1.50	3,100			3837.5	1087.5	2750.0

8.5.2 The Stage-Discharge Relationship

The discharge from a reservoir or detention facility depends on the depth of flow and the characteristics of the outlet facility. The outlet facility can include a weir, an orifice, or both. The weir and orifice equations were introduced in Section 8.3. For a given weir length or orifice area, the stage-discharge relationship can be computed directly from either Equation 8.10 or Equation 8.19.

8.6 DESIGN PROCEDURE

Two general types of hydrologic synthesis are used for storage routing methods. First, watershed studies are frequently conducted where structures currently exist. In this case of synthesis, the response of the system rather than the design of a structure is the objective. A design-flood hydrograph is routed through an existing structure using a known stage-storage-discharge relationship, with the output of the computation being a computed hydrograph.

Very often, watershed studies are undertaken to evaluate the hydrologic effects of various land use conditions, such as a natural state, current conditions, and completely developed, or for different zoning practices. For such watershed studies, the routing is not undertaken as part of the design of the structure.

The second case of hydrologic synthesis is the design of the storage and outlet facility. Quite often, the design of a storage facility centers on a target discharge. For example, in the design of urban detention basins, stormwater management policies may require the post-development peak discharge to be no greater than the peak discharge for the pre-development watershed conditions at a selected exceedence frequency (i.e., return period).

In designing the riser to meet the target discharge, the goal of the design is to identify the riser characteristics (weir length and/or area of the orifice) that will limit the computed post-development peak discharge out of the detention basin so that it does not exceed the target discharge, which is usually the peak discharge that would occur if development had not taken place within the watershed. The stage-storage-discharge relationship is a function of both the target discharge and the riser characteristics. Because of this interdependence, the design procedure is iterative. That is, the design procedure begins with some assumption about the riser characteristics. Then the storage routing procedure is applied to determine whether or not the target-discharge requirement has been met. The riser characteristics are continually modified until the target is met. If the weir length is too long or the orifice area too large, the target discharge will be exceeded. If the computed discharge is less than the target discharge (i.e., the weir length is too short or the orifice is too small), the design will result in storage that is greater than that really required.

The design procedure is summarized in the flowchart of Figure 8.10. There are four requirements for the design. First, initial conditions must be established; this includes setting the time interval Δt, the storm time at which computations end, and the initial outflow O_1 and storage S_1. Second, the design-storm inflow hydrograph and the target discharge q_o must be computed. The design-storm inflow hydrograph is usually the output from the convolution of a rainfall-excess hyetograph and a unit hydrograph, with the post-development conditions used to compute the rainfall excess. The target discharge is usually the peak discharge of the pre-development hydrograph. Third, the riser characteristics (i.e., number of stages, type of outlet, and values of the discharge coefficients) must be set. Fourth, topographic information must be obtained and the stage-storage relationship computed. These four inputs are indicated in Figure 8.10 by nodes A, B, C, and D, respectively.

The design process is iterative, with two loops. The interior loop, which begins at node F in Figure 8.10, uses the storage-indication routing method to route the design-storm inflow hydrograph through a storage structure that has an assumed design. The exterior loop, which begins at node E in Figure 8.10, uses assumed weir lengths or orifice areas and assumed elevations to compute the stage-discharge and storage-indication curves.

Based on these inputs and initial computations, the inflow hydrograph can be routed through the storage basin; this begins at node F in Figure 8.10. The storage-indication routing procedure was detailed in Section 7.2. Once the outflow hydrograph is computed, its peak discharge is compared to the target discharge q_o. If it is greater than q_o, then the capacity of the assumed outlet configuration is too large. Thus, the weir lengths or orifice areas should be decreased (return to node E in Figure 8.10). If the peak discharge of the outflow is less than the target discharge q_o, the assumed outlet configuration does not provide for sufficient outflow. The weir lengths or orifice areas can be increased (return to node E in Figure 8.10). This will require recomputing the stage-discharge relationship, the storage-indication curve, and the storage-discharge curve.

The routing process, which begins at node F of Figure 8.10, is then repeated with the new stage-storage-discharge information, and the maximum discharge of the new outflow hydrograph is compared to the target discharge. When the peak outflow approximately equals the target discharge, the assumed outlet facility is a reasonable design. The required storage is estimated by the largest value of storage S_2. The depth of storage is estimated by using the maximum S_2 as input to the stage-storage curve. The computed design should be evaluated for safety and cost.

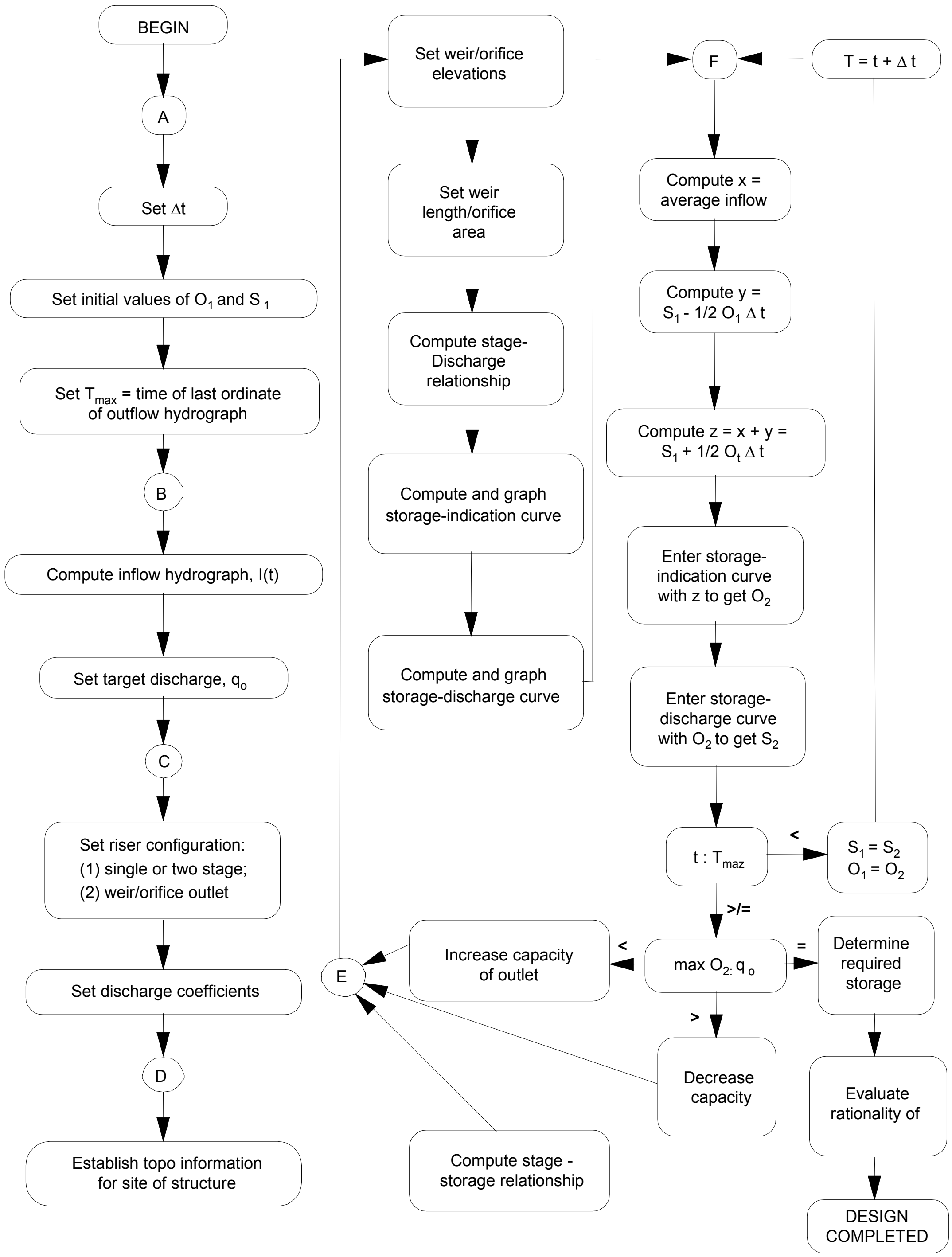


Figure 8.10. Flowchart of Storage Basin Design Procedure

Example 8.11(SI). A detailed example will be used to illustrate the detention basin design process. The watershed has a drainage area of 15.5 ha and is entirely in C soil. The design will use a 10-year exceedence frequency. In the pre-development condition, the watershed was 40 percent forest in good condition (CN = 70) and 60 percent in brush in good condition (CN = 65); therefore, the weighted CN is 67. For post-development conditions, the watershed has the following land cover: 26.1 percent in row houses (CN = 90); 36.6 percent in 0.1 ha lots (CN = 83); 13.0 percent in light commercial (75 percent impervious); and 24.3 percent in open space in good condition (CN = 74). The CN for the commercial area is 0.75(98) + 0.25(74) = 92. Thus, the weighted CN for the watershed is:

$$CN_a = [\ 0.261(90) + 0.366(83) + 0.130(92) + 0.243(74)\ ] = 83.8 \text{ (use 84)}$$

Characteristics of the principal flow paths for both watershed conditions are given in Table 8.5. For a 10-year exceedence frequency, assume that the 24-hour rainfall depth is 122 mm.

Table 8.5(SI). Computation of Times of Concentration

Condition	Land Cover	Length (m)	n	Slope (%)	Rainfall i (mm/h)	Hydraulic Radius (m)	Method	Travel Time (min)	t_c (min)
Before	Forest	46	0.3	2.0	127	-	Kinematic	15.7	
	Grassed swale	61	-	1.7	-	-	Eq. 2.5	1.7	
	Gully	107	-	1.5	-	-	Eq. 2.5	3.0	
	Open Channel	366	0.05	1.2	-	0.8	Manning's	7.2	27.6
After	Forest	15	0.3	2.0	179	-	Kinematic	7.0	
	Asphalt	30	0.012	2.0	190	-	Kinematic	1.5	
	Asphalt gutter	91	0.012	1.6	-	0.10	Manning's	1.6	
	Circular pipe	183	0.024	1.4	-	0.25	Manning's	3.3	
	Open channel	229	0.04	1.3	-	1.00	Manning's	3.0	16.4

Using the pre- and post-development CNs, the excess rainfall depths are computed from Equation 5.18 as 42.4 mm and 78.5 mm, respectively. The initial abstractions are 22.7 and 9.7 mm, which yield I_a/P ratios of 0.21 and 0.08 (use 0.1) for pre- and post-development, respectively. Using the times of concentration and I_a/P ratios, unit peak discharges of 0.215 and 0.306 $m^3/s/km^2/mm$ are taken from Equation 5.22 for the pre- and post-development conditions, respectively. These yield peak discharges of:

$$q_{pb} = q_{ub} Q_b A = 0.215\,(0.155)(42.4) = 1.41\ \ m^3/s$$
$$q_{pa} = q_{ua} Q_a A = 0.306\,(0.155)(78.5) = 3.72\ \ m^3/s$$

Thus, development within the watershed increased the 10-year peak discharge by 164 percent. The detention basin must be designed to reduce the peak discharge from 3.72 m³/s to 1.41 m³/s.

The stage-storage relationship is computed from topographic information. At the site where the detention structure will be located, surface areas were estimated from the topographic map at an increment of 0.15 m (see Table 8.5(SI)). The product of the average area (column 3) and the incremental depth of 0.15 m yield the incremental storage (column 4). The storage for any stage is the accumulated incremental storage. A graphical presentation of columns 1 and 5 would be the stage-storage relationship. The values were used to fit the following power model:

$$V_s = 3{,}256\ h^{1.027}$$

in which V_s is the storage (m^3) and h is the depth (m).

For design, an input hydrograph is necessary. The ordinates on a 0.1-hour increment are used for a 2-hour period of the 24-hour storm; discharges for the remainder of the 24-hour storm duration were either zero or very small.

The design policy requires a one-stage riser with a weir. An inflow hydrograph will be routed using the storage-indication method. An initial estimate of the weir length is made using the weir equation, with an assumed depth of 1.5 m:

$$L_w = \left(\frac{1}{\sqrt{2g}\,C_w}\right)\frac{q_o}{h^{1.5}} = \left(\frac{1}{1.705(0.94)}\right)\frac{1.41}{(1.5)^{1.5}} = 0.5\ \ m$$

Thus, an initial weir length of 0.5 m will be used. The stage-storage-discharge relationship and the storage-indication curve (($S/\Delta t$ + O/2) versus discharge) are given in Table 8.6(SI).

The inflow hydrograph is routed with the storage-indication method; the results are given in Table 8.7(SI). The largest outflow occurred at 1.7 hours. The peak outflow is 1.30 m³/s, which is less than the allowable peak of 1.41 m³/s. The maximum storage is 4,520 m^3.

Since the computed peak is less than the allowable peak, the required storage volume can be reduced by allowing a higher peak discharge to leave the basin. Thus, the weir length is increased to 0.6 m and the computations repeated. Since the weir length is changed, the stage-discharge and storage-indication curves must be recomputed (see Table 8.8(SI)). The inflow hydrograph is routed (see Table 8.9(SI)) with a resulting peak discharge of 1.45 m³/s. This exceeds the allowable peak of 1.41 m³/s.

At this point, it is appropriate to evaluate whether or not an additional iteration on weir length is worthwhile, especially in the context of available constructed weir lengths. The 0.5 m length meets our design requirement; the 0.6 m length does not. If an intermediate length is constructible, further analysis may be warranted. If not, the 0.5 m length must be selected.

Table 8.6(SI). Derivation of Stage-Storage Relationship

(1) Depth, h (m)	(2) Surface Area (m^2)	(3) Average Area (m^2)	(4) Incremental Storage (m^3)	(5) Storage Volume, V_s (m^3)
0.00	3,110			0
		3,125	469	
0.15	3,140			469
		3,165	475	
0.30	3,190			944
		3,200	480	
0.45	3,210			1,424
		3,245	487	
0.60	3,280			1,911
		3,303	495	
0.75	3,320			2,406
		3,340	501	
0.90	3,360			2,907
		3,390	509	
1.05	3,420			3,416
		3,450	518	
1.20	3,480			3,934
		3,505	526	
1.35	3,530			4,460
		3,565	535	
1.50	3,600			4,995

Table 8.7(SI). Stage-Storage-Discharge and Storage-Indication Curves: Weir Length, 0.5 m

(1) Depth (m)	(2) Storage (m^3)	(3) Discharge (m^3/s)	(4) ($S/\Delta t + O/2$) (m^3/s)
0.00	0	0.00	0.00
0.15	464	0.05	1.31
0.30	946	0.13	2.69
0.45	1,434	0.24	4.10
0.60	1,927	0.37	5.54
0.75	2,423	0.52	6.99
0.90	2,922	0.68	8.46
1.05	3,423	0.86	9.94
1.20	3,926	1.05	11.43
1.35	4,431	1.26	12.94
1.50	4,938	1.47	14.45
1.65	5,446	1.70	15.98
1.80	5,955	1.94	17.51
1.95	6,465	2.18	19.05
2.10	6,976	2.44	20.60
L (m) = 0.5			

Table 8.8(SI). Storage-Indication Routing: Weir Length, 0.5 m

Time (h)	Inflow (m^3/s)	Average inflow (m^3/s)	($S/\Delta t$ +O/2) (m^3/s)	O (m^3/s)	S (m^3)
0.0	0.00	0.015	0.00	0.000	0
0.1	0.03	0.045	0.02	0.001	5
0.2	0.06	0.070	0.06	0.002	21
0.3	0.08	0.095	0.13	0.005	45
0.4	0.11	0.125	0.22	0.008	77
0.5	0.14	0.155	0.34	0.012	119
0.6	0.17	0.185	0.48	0.017	169
0.7	0.20	0.225	0.65	0.023	229
0.8	0.25	0.325	0.85	0.030	300
0.9	0.40	0.495	1.14	0.041	404
1.0	0.59	0.890	1.60	0.064	564
1.1	1.19	1.800	2.42	0.115	852
1.2	2.41	3.030	4.11	0.242	1,435
1.3	3.65	3.680	6.90	0.511	2,391
1.4	3.71	3.130	10.07	0.878	3,465
1.5	2.55	2.055	12.32	1.173	4,223
1.6	1.56	1.305	13.20	1.294	4,519
1.7	1.05	0.905	13.21	1.296	4,522
1.8	0.76	0.665	12.82	1.241	4,392
1.9	0.57	0.495	12.24	1.163	4,198
2.0	0.42	0.395	11.58	1.073	3,974
2.1	0.37		10.90	0.985	3,746
L (m) = 0.5					

Table 8.9(SI). Stage-Storage-Discharge and Storage-Indication Curves: Weir Length, 0.6 m

(1) Depth (m)	(2) Storage (m³)	(3) Discharge (m³/s)	(4) (S/ Δt + O/2) (m³/s)
0.00	0	0.00	0.00
0.15	464	0.06	1.32
0.30	946	0.16	2.71
0.45	1,434	0.29	4.13
0.60	1,927	0.45	5.58
0.75	2,423	0.62	7.04
0.90	2,922	0.82	8.53
1.05	3,423	1.03	10.03
1.20	3,926	1.26	11.54
1.35	4,431	1.51	13.06
1.50	4,938	1.77	14.60
1.65	5,446	2.04	16.15
1.80	5,955	2.32	17.70
1.95	6,465	2.62	19.27
2.10	6,976	2.93	20.84
L (m) = 0.6			

Table 8.10(SI). Storage-Indication Routing: Weir Length, 0.6 m

Time (h)	Inflow (m^3/s)	Average Inflow (m^3/s)	($S/\Delta t + O/2$) (m^3/s)	O (m^3/s)	S (m^3)
0.0	0.00	0.015	0.00	0.000	0
0.1	0.03	0.045	0.02	0.001	5
0.2	0.06	0.070	0.06	0.003	21
0.3	0.08	0.095	0.13	0.005	45
0.4	0.11	0.125	0.22	0.009	76
0.5	0.14	0.155	0.33	0.014	117
0.6	0.17	0.185	0.47	0.020	167
0.7	0.20	0.225	0.64	0.027	225
0.8	0.25	0.325	0.84	0.035	295
0.9	0.40	0.495	1.13	0.048	397
1.0	0.59	0.890	1.57	0.075	553
1.1	1.19	1.800	2.39	0.135	835
1.2	2.41	3.030	4.05	0.283	1,408
1.3	3.65	3.680	6.80	0.595	2,341
1.4	3.71	3.130	9.89	1.014	3,376
1.5	2.55	2.055	12.00	1.338	4,079
1.6	1.56	1.305	12.72	1.453	4,317
1.7	1.05	0.905	12.57	1.429	4,268
1.8	0.76	0.665	12.05	1.345	4,094
1.9	0.57	0.495	11.37	1.238	3,869
2.0	0.42	0.395	10.62	1.125	3,622
2.1	0.37		9.89	1.016	3,379
L (m) = 0.6					

Example 8.11(CU). A detailed example will be used to illustrate the detention basin design process. The watershed has a drainage area of 38 acres and is entirely in C soil. The design will use a 10-year exceedence frequency. In the pre-development condition, the watershed was 40 percent forest in good condition (CN = 70) and 60 percent in brush in good condition (CN = 65); therefore, the weighted CN is 67. For post-development conditions, the watershed has the following land cover: 26.1 percent in row houses (CN = 90); 36.6 percent in ¼ acre lots (CN = 83); 13.0 percent in light commercial (75 percent impervious); and 24.3 percent in open space in good condition (CN = 74). The CN for the commercial area is 0.75(98) + 0.25(74) = 92. Thus, the weighted CN for the watershed is:

$$CN_a = [\ 0.261(90) + 0.366(83) + 0.130(92) + 0.243(74)\ ] = 83.8 \text{ (use 84)}$$

Characteristics of the principal flow paths for both watershed conditions are given in Table 8.5(CU). For a 10-year exceedence frequency, assume that the 24-hour rainfall depth is 4.8 in.

Table 8.5(CU). Computation of Times of Concentration

Condition	Land Cover	Length (ft)	n	Slope (%)	Rainfall i (in/h)	Hydraulic Radius (ft)	Method	Travel Time (min)	t_c (min)
Before	Forest	150	0.3	2.0	5.0	-	Kinematic	15.7	
	Grassed swale	200	-	1.7	-	-	Eq. 2.5	1.7	
	Gully	350	-	1.5	-	-	Eq. 2.5	3.0	
	Open Channel	1,200	0.05	1.2	-	2.6	Manning's eq.	7.2	27.6
After	Forest	50	0.3	2.0	7.0	-	Kinematic	7.0	
	Asphalt	100	0.012	2.0	7.5	-	Kinematic	1.5	
	Asphalt gutter	300	0.012	1.6	-	0.33	Manning's eq.	1.6	
	Circular pipe	600	0.024	1.4	-	0.82	Manning's eq.	3.3	
	Open channel	750	0.04	1.3	-	3.28	Manning's eq.	3.0	16.4

Using the pre- and post-development CNs, the excess rainfall depths are computed from Equation 5.18 as 1.67 in and 3.09 in, respectively. The initial abstractions are 0.99 and 0.38 in, which yields I_a/P ratios of 0.21 and 0.08 (use 0.1) for pre- and post-development, respectively. Using the times of concentration and I_a/P ratios, unit peak discharges of 501 and 716 $ft^3/s/mi^2/in$ are taken from Equation 5.22 for the pre- and post-development conditions, respectively. These yield peak discharges of:

$$q_{pb} = q_{ub}\, Q_b\, A = 501\ (1.67)\ (38/640) = 50\ ft^3/s$$
$$q_{pb} = q_{ub}\, Q_b\, A = 716\ (3.09)\ (38/640) = 131\ ft^3/s$$

Thus, development within the watershed increased the 10-year peak discharge by 162 percent. The detention basin must be designed to reduce the peak discharge from 131 ft^3/s to 50 ft^3/s.

The stage-storage relationship is computed from topographic information. At the site where the detention structure will be located, surface areas were estimated from the topographic map at an increment of 0.5 ft (see Table 8.6(CU)). The product of the average area (column 3) and the incremental depth of 0.5 ft yield the incremental storage (column 4). The storage for any stage is the accumulated incremental storage. A graphical presentation of columns 1 and 5 would be the stage-storage relationship. The values were used to fit the following power model:

$$V_s = 34\ h^{1.027}$$

in which V_s is the storage (ft^3) and h is the depth (ft).

For design, an input hydrograph is necessary. The ordinates on a 0.1-hour increment are used for a 2-hour period of the 24-hour storm; discharges for the remainder of the 24-hour storm duration were either zero or very small.

The design policy requires a one-stage riser with a weir. An inflow hydrograph will be routed using the storage-indication method. An initial estimate of the weir length is made using the weir equation, with an assumed depth of 4.9 ft:

$$L_w = \left(\frac{1}{\sqrt{2g}\,C_w}\right)\frac{q_o}{h^{1.5}} = \left(\frac{1}{3.09(0.94)}\right)\frac{50}{(4.9)^{1.5}} = 1.6\ ft$$

Thus, an initial weir length of 1.6 ft will be used. The stage-storage-discharge relationship and the storage-indication curve ($(S/\Delta t + O/2)$ versus discharge) are given in Table 8.7(CU).

The inflow hydrograph is routed with the storage-indication method; the results are given in Table 8.8(CU). The largest outflow occurred at 1.7 hours. The peak outflow is 45 ft^3/s, which is less than the allowable peak of 50 ft^3/s. The maximum storage is 161,000 ft^3.

Since the computed peak is less than the allowable peak, the required storage volume can be reduced by allowing a higher peak discharge to leave the basin. Thus, the weir length is increased to 2.0 ft and the computations repeated. Since the weir length is changed, the stage-discharge and storage-indication curves must be recomputed (see Table 8.8(CU)). The inflow hydrograph is routed (see Table 8.10(CU)) with a resulting peak discharge of 52 ft^3/s. This exceeds the allowable peak of 50 ft^3/s.

At this point, it is appropriate to evaluate whether or not an additional iteration on weir length is worthwhile, especially in the context of available constructed weir lengths. The 1.6 ft length meets our design requirement; the 2.0 ft length does not. If an intermediate length is constructible, further analysis may be warranted. If not, the 1.6 ft length must be selected.

Table 8.6(CU). Derivation of Stage-Storage Relationship

(1) Depth, h (ft)	(2) Surface Area, A (ft^2)	(3) Average Area (ft^2)	(4) Incremental Storage (ft^3)	(5) Storage Volume, V_s (ft^3)
0.00	33,500			0
		33,650	16,825	
0.50	33,800			16,825
		34,050	17,025	
1.00	34,300			33,850
		34,450	17,225	
1.50	34,600			51,075
		34,950	17,475	
2.00	35,300			68,550
		35,500	17,750	
2.50	35,700			86,300
		35,950	17,975	
3.00	36,200			104,275
		36,500	18,250	
3.50	36,800			122,525
		37,150	18,575	
4.00	37,500			141,100
		37,750	18,875	
4.50	38,000			159,975
		38,400	19,200	
5.00	38,800			179,175

Table 8.7(CU). Stage-Storage-Discharge and Storage-Indication Curves: Weir Length, 1.6 ft

(1) Depth (ft)	(2) Storage (ft^3)	(3) Discharge (ft^3/s)	(4) ($S/\Delta t + O/2$) (ft^3/s)
0.00	0	0.0	0
0.50	16,700	1.6	47
1.00	34,000	4.6	97
1.50	51,600	8.5	148
2.00	69,300	13.1	199
2.50	87,100	18.4	251
3.00	105,100	24.1	304
3.50	123,100	30.4	357
4.00	141,200	37.2	411
4.50	159,300	44.3	465
5.00	177,600	51.9	519
5.50	195,800	59.9	574
6.00	214,100	68.3	629
6.50	232,500	77.0	684
7.00	250,800	86.0	740
L (ft) = 1.6			

Table 8.8(CU). Storage-Indication Routing: Weir Length, 1.6 ft

Time (h)	Inflow (ft^3/s)	Average Inflow (ft^3/s)	($S/\Delta t$ +O/2) (ft^3/s)	O (ft^3/s)	S (ft^3)
0.0	0		0.0	0.0	0
		0.5			
0.1	1		0.5	0.0	200
		1.5			
0.2	2		2.0	0.1	700
		2.5			
0.3	3		4.4	0.2	1,600
		3.5			
0.4	4		7.8	0.3	2,700
		4.5			
0.5	5		12.0	0.4	4,200
		5.5			
0.6	6		17.1	0.6	6,000
		6.5			
0.7	7		23.0	0.8	8,100
		8.0			
0.8	9		30.2	1.0	10,700
		11.5			
0.9	14		40.6	1.4	14,400
		17.5			
1.0	21		56.7	2.2	20,000
		31.5			
1.1	42		86.0	4.0	30,200
		63.5			
1.2	85		145.5	8.4	50,900
		107.0			
1.3	129		244.1	17.7	84,700
		130.0			
1.4	131		356.5	30.3	122,900
		110.5			
1.5	90		436.6	40.6	149,900
		72.5			
1.6	55		468.5	44.9	160,600
		46.0			
1.7	37		469.6	45.0	161,000
		32.0			
1.8	27		456.6	43.3	156,600
		23.5			
1.9	20		436.8	40.6	149,900
		17.5			
2.0	15		413.7	37.6	142,200
		14.0			
2.1	13		390.1	34.6	134,200
L (ft) = 1.6					

Table 8.9(CU). Stage-Storage-Discharge and Storage-Indication Curves: Weir Length, 2.0 ft

(1) Depth (ft)	(2) Storage (ft^3)	(3) Discharge (ft^3/s)	(4) ($S/\Delta t + O/2$) (ft^3/s)
0.00	0	0.0	0
0.50	16,700	2.1	47
1.00	34,000	5.8	97
1.50	51,600	10.7	149
2.00	69,300	16.4	201
2.50	87,100	23.0	253
3.00	105,100	30.2	307
3.50	123,100	38.0	361
4.00	141,200	46.5	415
4.50	159,300	55.4	470
5.00	177,600	64.9	526
5.50	195,800	74.9	581
6.00	214,100	85.3	637
6.50	232,500	96.2	694
7.00	250,800	107.5	750
L (ft) = 2.0			

Table 8.10(CU). Storage-Indication Routing: Weir Length, 2.0 ft

Time (h)	Inflow (ft^3/s)	Average Inflow (ft^3/s)	$(S/\Delta t + O/2)$ (ft^3/s)	O (ft^3/s)	S (ft^3)
0.0	0		0.0	0.0	0
		0.5			
0.1	1		0.5	0.0	200
		1.5			
0.2	2		2.0	0.1	700
		2.5			
0.3	3		4.4	0.2	1,500
		3.5			
0.4	4		7.7	0.3	2,700
		4.5			
0.5	5		11.9	0.5	4,200
		5.5			
0.6	6		16.9	0.7	5,900
		6.5			
0.7	7		22.6	1.0	8,000
		8.0			
0.8	9		29.6	1.3	10,400
		11.5			
0.9	14		39.9	1.7	14,000
		17.5			
1.0	21		55.6	2.7	19,500
		31.5			
1.1	42		84.5	4.8	29,500
		63.5			
1.2	85		143.1	10.1	49,700
		107.0			
1.3	129		240.0	21.3	82,600
		130.0			
1.4	131		348.7	36.2	119,000
		110.5			
1.5	90		423.0	47.7	143,700
		72.5			
1.6	55		447.8	51.8	151,900
		46.0			
1.7	37		442.0	50.8	150,000
		32.0			
1.8	27		423.2	47.7	143,800
		23.5			
1.9	20		399.0	43.9	135,700
		17.5			
2.0	15		372.6	39.8	127,000
		14.0			
2.1	13		346.8	36.0	118,400
L (ft) = 2.0					

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CHAPTER 9

SPECIAL TOPICS IN HYDROLOGY

Hydrologic engineers have become increasingly proficient in providing design information for the proper sizing of storm drainage and hydraulic structures using the techniques presented in previous chapters. However, they are often faced with design challenges which are not adequately addressed, nor anticipated, by standard hydrologic procedures. This chapter addresses three unique situations where engineers must go beyond standard methodologies: wetlands, snowmelt, and arid lands hydrology. This chapter also briefly addresses advanced applications in hydrology, including use of geographic information systems (GIS).

9.1 WETLANDS

Wetlands mitigation analysis and design is a complex topic most successfully conducted with an interdisciplinary team including, at a minimum, a wetlands scientist and a hydrologic engineer. This section addresses the role of the hydrologic engineer and the tools needed by the engineer for wetlands mitigation projects. This is not to minimize the role of the wetland scientist, rather it is only a recognition that the knowledge and tools brought to the mitigation process by the wetland scientist are beyond the scope of this discussion.

9.1.1 Wetland Fundamentals

The engineer must have a basic understanding of what wetlands are and how they function within a natural ecosystem to effectively contribute to the analysis or design process for wetland mitigation projects. This includes knowing the definition of a wetland, its functions and values, various wetland types, and the concept of hydroperiod.

9.1.1.1 Definition

The definition of a wetland has evolved over the years and has taken many forms. The variety of types of wetlands throughout the country has further complicated development of a universally accepted definition. Two Federal agencies with specific responsibilities for the regulation of wetlands have agreed to and codified the following wetland definition:

> "those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions."
> (EPA, 40 CFR 230.3 and Corps of Engineers 33 CFR 328.3)

This definition is applied in the field by a wetlands scientist to determine the existence and extent of a wetland. This is accomplished through a process of wetland delineation, which generally must be adopted by the appropriate regulatory authority (state or Federal). Once the location of a wetland is determined, the potential effects of a transportation project on the wetland may be evaluated and, when appropriate, mitigated.

A "jurisdictional" wetland is a delineated wetland considered to be subject to the regulatory jurisdiction of the U.S. Army Corps of Engineers under the authority of section 404 of the Clean

Water Act. Three conditions derived from the definition of a wetland are required for a jurisdictional wetland:

1. Hydrophytic vegetation dominant: vegetation adapted to wet conditions
2. Hydric soils: soil types developed under at least periodically wet conditions
3. Saturation at or near the surface, or inundation for some period of time during an average year.

All three criteria must be satisfied for a wetland, though the presence of the appropriate hydrology will ultimately permit the development of hydrophytic vegetation and hydric soils over time. For this reason naturally or artificially induced changes to hydrology that increase the amount of water at a site over time will attract hydrophytic vegetation and create the conditions for the development of hydric soils, thereby creating wetlands. This process has been observed, in its unintentional form, within stormwater management facilities. Unintentionally created wetlands have attracted regulatory oversight, which in turn has created maintenance issues in some locations.

A key challenge in the delineation, analysis, and design of wetlands is the quantification of many of the terms used to define wetlands, such as: “periodically,” “some period of time,” and “average year.” The wetland scientist plays a key role in defining the needs of the desired vegetative communities while the hydrologic engineer is responsible for analyzing and designing a system to deliver the needed water.

9.1.1.2 Functions and Values

Effective wetland mitigation goes beyond meeting the vegetative, soil, and hydrologic conditions. Wetlands are diverse in their character and function within an ecosystem. If an existing wetland must be affected by a transportation project, mitigation must consider the existing functions of the wetland as well as the value placed on those functions so those functions and values may be transferred to another location or replaced by equivalent functions and values. A functional assessment is the mechanism for determining the functions of an existing wetland.

Wetland functions are the objective activities or natural processes performed by wetlands. Values are the worth placed on those functions by human society. Values, therefore, are subjective and may change from group to group, place to place, and over time.

Wetland functions may be categorized in three general groups: ecological, economic, and recreational/aesthetic (as summarized below). Degraded wetlands may perform minimal function and therefore offer little value. Productive wetlands may offer multiple functions. Wetland functions include:

1. Ecological
    a. floodwater storage and detention
    b. water quality
    c. fish and wildlife habitat and food
2. Economic
    a. farming
    b. timber harvest
    c. special products
    d. recreation revenue

3. Recreation/Aesthetic
    a. hunting and trapping
    b. boating
    c. bird watching
    d. photography
    e. fishing and clamming

The diversity of wetland functions and the challenges of successfully mitigating the destruction of existing wetlands lead to a hierarchy of options in response to the potential loss of wetlands. Mitigation alternatives include – in preferential order – avoidance, minimization, and compensation. Avoidance of existing wetlands is preferred since it leaves the wetland and its functions unchanged. The overall mitigation preference hierarchy is shown in Figure 9.1.

When compensation is the only mitigation alternative, several means of compensation are available including creation of new wetlands, restoration of degraded wetlands, or enhancement of existing wetlands. Of the compensation options, restoration and enhancement are frequently preferred because of the higher probability of success.

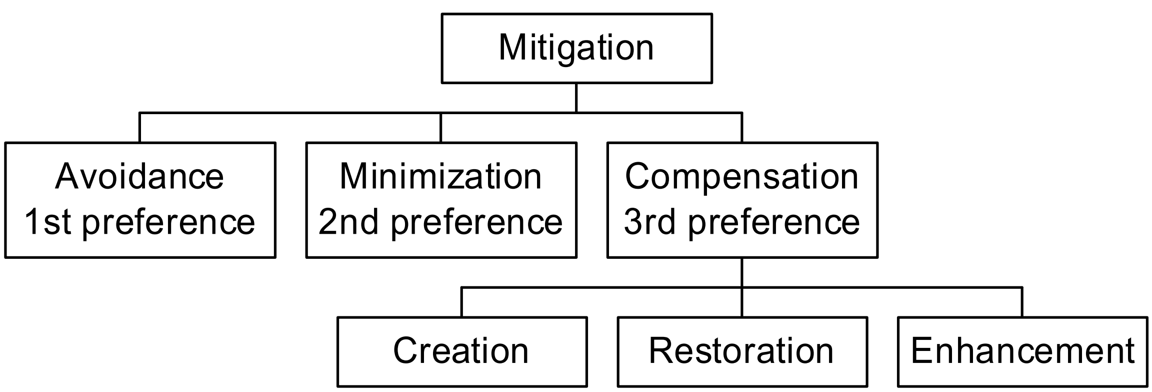


Figure 9.1. Wetland mitigation preferences

9.1.2 Wetland Types

Wetland mitigation requires a thorough understanding the type of wetland being mitigated so that an appropriate replacement may be designed. Wetlands vary in their sources of water, geology, morphology, and topography. Figures 9.2, 9.3, and 9.4 are photos of significantly different wetland types.

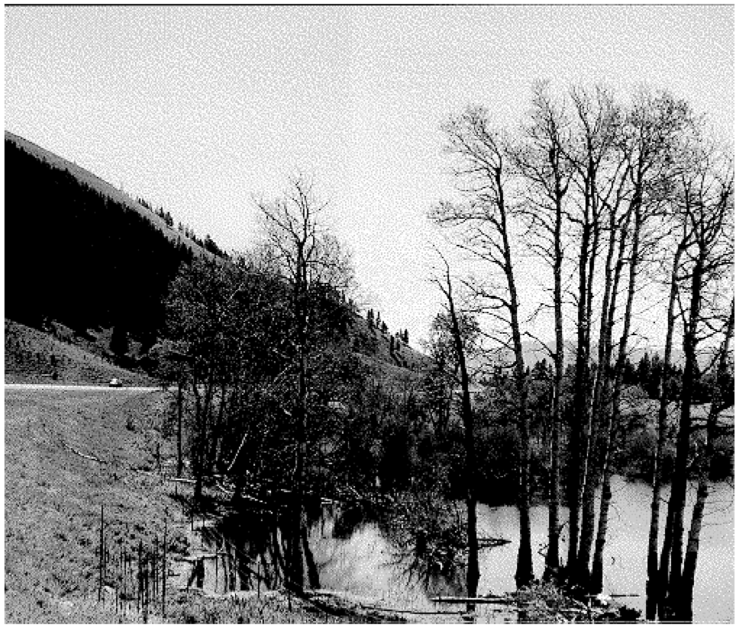


Figure 9.2. Blackfoot Waterfowl Area, Montana

Figure 9.3. Company Swamp, North Carolina

Figure 9.4. Highway 89, California

Several wetland classification schemes have been developed and applied over the years and they continue to evolve as more is learned about wetlands and their function in the environment. The Cowardin and Hydrogeomorphic (HGM) methods are the two most common schemes in use today. The U.S. Fish and Wildlife Service (Cowardin, et al., 1979) developed a classification system that focuses on mapping wetland types and determining how the ecology of the wetland fits into the surrounding ecosystem. Though it incorporates some aspects of hydrology and vegetation they are not primary considerations. The Hydrogeomorphic Method (HGM), developed by Brinson (1993), classifies wetland by geomorphic setting, water source, and hydrodynamics. Brinson's five HGM classes are riverine, fringe, depressional, slope, and extensive peatlands.

Detailed descriptions of these classification schemes are beyond the scope of this discussion. However, the engineer should not overlook the great variety in wetlands and work with a wetland scientist in properly classifying wetlands.

9.1.3 Hydroperiod

The primary focus for the hydrologic engineer in wetland mitigation is assessing the hydroperiod for existing wetlands and designing created or restored wetlands to achieve a specific hydroperiod. Hydroperiod is the extent and duration of inundation and/or saturation of wetland systems. Stormwater wetlands tend to have a hydroperiod characterized by frequent to chronic inundation by standing water. Generally, the concern is having too little water when needed though it is possible to have too much water for the desired vegetation.

The concept of hydroperiod is further complicated by consideration of the growing season. The growing season is the period of most active growth for wetland vegetation. In areas of the country that experience freezes and thaws, it is the period between the last freeze in the spring and the first frost in the fall for the freeze threshold temperature of the vegetation.

The variety of hydroperiods is best illustrated through several examples depicting variation of inundation depth versus time over the course of a year. The prairie pothole in North Dakota (Figure 9.5) shows depths ranging from no standing water in the Fall and Winter to greater depths in the Spring and Summer. The northern riparian wetland in Ohio (Figure 9.6) is representative of a wetland adjacent to a stream or river that serves as its primary source of water. Water level in the wetland is closely linked to discharge conditions in the stream. The tidal marsh in Delaware (Figure 9.7) displays a water level variations derived from the adjacent tidal variations.

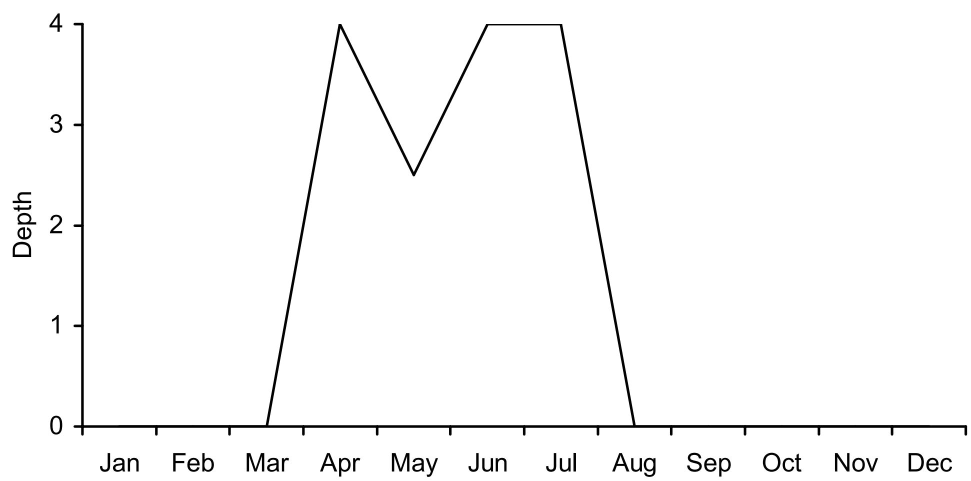


Figure 9.5. Prairie Pothole, North Dakota

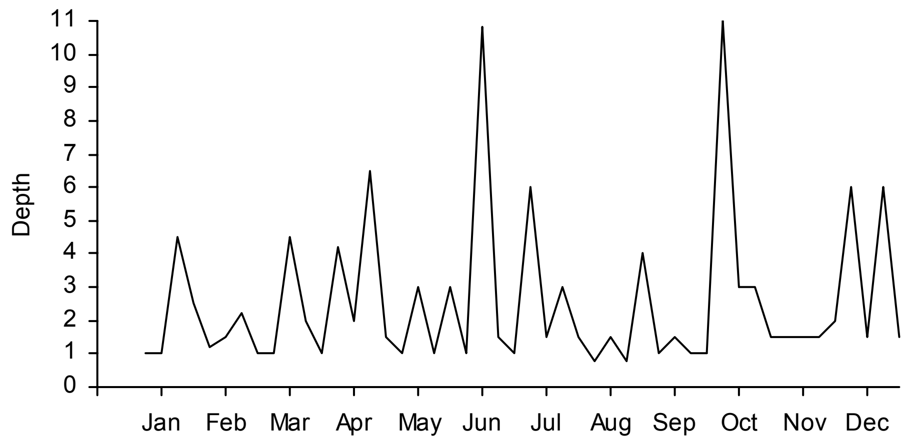


Figure 9.6. Northern Riparian, Ohio

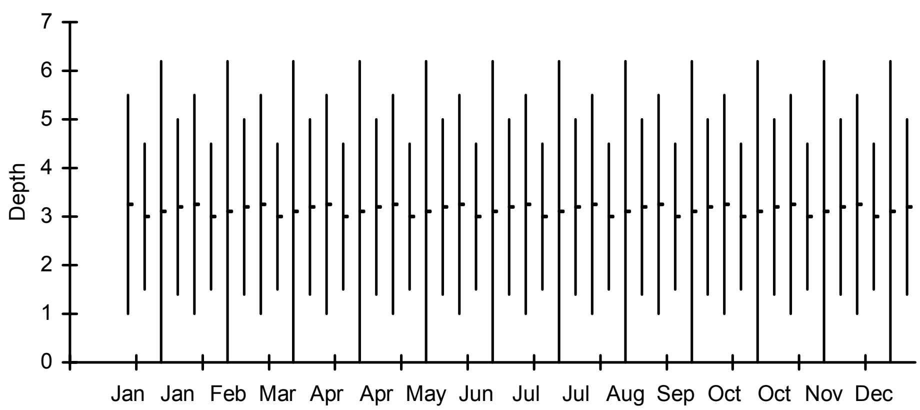


Figure 9.7. Tidal Marsh, Delaware

The design hydroperiod for wetland mitigation is largely determined by the type of vegetation. The wetlands specialist will assist the engineer in determining the desired depths, durations, and timing desired for establishing the design goals. These goals may vary widely. For example, in a Palustrine Marsh inundation should be at least 200 mm (8 in) at least 8 months of the year, on average. (Palustrine wetlands are all nontidal wetlands dominated by trees, shrubs, persistent emergents, emergent mosses or lichens and all tidal wetlands where salinity from ocean-derived salts is less than 0.5%.) Other wetlands may specify saturation at or near the ground surface for 7-10 consecutive days during the growing season in most years. Table 9.1 provides a summary of inundation requirements for selected vegetation.

Table 9.1. Selected Depth Requirements

Plant Growth Form	Average Water Depth (mm)	Average Water Depth (in)
Submergents, e.g. water celery	>500	>20
Floating leaves, e.g. water lily	200-1000	8-39
Herbaceous emergents, e.g. bulrushes	0-500	0-20
Shrubs, e.g. Buttonbush	0-200	0-8
Trees, e.g. Cypress	0-500	0-20

9.1.4 Wetland Banking

Wetland mitigation through creation of new wetlands and restoration of impaired wetlands is complicated by the variety of factors that determine wetland viability. One option for wetland mitigation is the purchase of credits at a wetland bank. A wetland bank is a location where wetlands have been created and maintained in anticipation of the needs of others to purchase credits for wetland losses elsewhere. If for example, a highway project unavoidably destroys 1 ha of wetlands. Compensation may be made through the purchase of credits at an approved wetland banking site. Wetland banking is generally the most expedient approach to wetland mitigation and may also be the most cost-effective in many cases.

9.1.5 Models of Wetland Creation and Restoration

The combination of expertise and information required to successfully design and construct creation and/or restoration projects is substantial. To assist in organizing the available data and establishing goals for wetland mitigation projects, several types of conceptual wetland models have been created. These models describe the general characteristics of the desired wetland for a given site and provide guidance on how to approach the design task. The models, therefore, provide the basis for engineering wetlands.

Wetland models are derived from classification systems that explicitly address hydrology such as the HGM method. They specify the source or sources of water, suggest water control structure design, characterize wetland setting, and guide hydroperiod criteria selection. These models describe the general characteristics of the desired wetland for a given site. It also provides guidance on how to approach the design task. Three major wetland model groups are surface water, groundwater, and enhancement restoration models.

Although these wetland restoration model groups are not mutually exclusive, the surface water models are mainly applicable to wetland creation rather than enhancement or restoration. As the name implies, surface water models rely on surface sources of water for the wetland. Surface water models are further subdivided to inline stream, offline stream, and surface categories based on how flows are brought to the site. The inline stream flow model requires that a water control structure be placed in the stream. It is appropriate only where low flows and limited debris occur, specifically, low energy locations. Inline wetland systems are difficult to implement from a regulatory perspective because blocking a stream is generally discouraged. The offline stream flow model also uses a water control structure, but in this case to divert flows to an offline location. This approach is advantageous in higher energy streams, but controlling erosion is a key to success. Finally, the surface flow model is based on intercepting surface runoff through the use of berms and excavation.

Groundwater models rely primarily on groundwater sources for water supply, but should be supplemented with surface water whenever possible. Groundwater models are generally grouped in two categories, spring/seepage flow and groundwater interception. Spring/seepage wetlands are created by excavating below a groundwater source and, possibly, using berms for containment. Surface water sources often supplement this type of wetland creation. Groundwater interception is also based on excavation to a depth below the groundwater table. The design challenge in this case is properly characterizing the behavior of the water table over the long term with generally short periods of monitoring observations. Efforts to create a wetland based on groundwater interception have observed the highest failure rates.

For both groundwater and surface water systems, a reliable source of water is most important for a basin wetland. Basin wetlands are not directly connected to a surface water source such as a stream, river, lake, reservoir, or estuary. For all created wetlands, great care is required to understand the source of both ground and surface water so that the effects of changes in land use on water availability can be anticipated.

The enhancement/restoration models build on existing water bodies and/or existing wetlands. The shared water supply model is based on extending the use of water provided to an existing wetland by constructing additional wetlands to expand the area. Care must be taken to not dry out the existing wetland by expanding the wetland area. While the shared water supply model seeks to expand area, the aquatic bed model modifies the depths in an existing wetland to achieve alternative vegetative patterns and, in turn, alternative wetland functions and values.

The final three enhancement/restoration models build onto existing water bodies. The lake shore, island, and riparian rehabilitation models derive their sources of water from the adjacent water body, but modify depths to allow appropriate wetland vegetation to be established.

The design and use of a control structure may be required to deliver and/or retain water for any of the wetland models described. Control structure functions may include:

1. Control design depths (minimum and maximum)
2. Distribute flows
3. Provide overflow capability

The potential physical configurations can take a variety of forms depending on the site. For controlling water levels, they may include headgates, pipes/culverts, flashboard culverts, weirs, and stoplog structures. For distributing flow over a wide area within a wetland or to different wetland cells, they may include distribution headers, swales, flow splitters, and baffles/finger dikes. In almost all cases, it is important that control structures be designed with adjustable features. This will permit changes to the structure after wetland construction to reflect observed versus anticipated water supply patterns, as well as providing a means for adjustment during extreme wet or dry years. Consideration is also required for potential erosion, overflow, and seepage. Additional information on design of water control structures may be found in a variety of sources including the Wetlands Engineering Handbook (Olin, et al, 2000).

9.1.6 Water Budgets

The key tool for the hydrologic engineer in designing a successful wetland is the water budget. The water budget is an analytical framework for accounting for the inflow, storage, and outflow of water at a given site. Within the water budget, the quantitative availability of water for the wetlands is determined along with its depth, duration, and frequency. The results of the water budget analysis are used to determine if the desired vegetation and wetland characteristics can be supported.

The continuity (i.e., storage routing) equation, as shown in Equation 9.1 is the foundation for performing a water budget. Conceptually, the procedure is simple: account for the inflows and outflows to the wetland mitigation site resulting in an understanding of the storage of water at the site. The challenge is adequately defining and quantifying the components of the water budget.

$$I - O = \frac{dS}{dt} \quad (9.1)$$

where,

I = water inflow
O = water outflow
dS = change in storage
dt = change in time

The components of a water budget include its inflows, outflows, and storage characteristics. The choice of wetland model will influence which of the inflows and outflows are included explicitly in the water budget.

9.1.6.1 Water Budget Inflows

Inflows include direct precipitation (rain and snow), surface water inflow (base flow and storm runoff), and groundwater inflow. Wetland failures rarely occur because too much water is available, though it is possible. Therefore, it is generally appropriate to ignore minor inputs and focus on estimating the major source of water. Such an approach is a conservative assumption that may increase the viability of a wetland.

Direct precipitation, that is, precipitation (rain and snow) falling directly on the surface area of the wetland, provides a source of water to the wetland. However, in heavily vegetated wetlands estimates of interception range from 0 to 35%. Frequently, direct precipitation may be ignored in water budgets because it is small compared to other water sources. Care should be taken to avoid double counting direct precipitation on the wetland and stormwater runoff flowing to the wetland.

Surface water inflow may be in the form of base flow in a stream (potentially fed by groundwater) or in direct runoff from precipitation in the contributing watershed of the wetland. Ideally, a long-term stream gage record of daily or hourly flows will exist at a site to determine surface water contributions. If a stream gage does not exist, the next preference is for a long-term precipitation record providing input to a calibrated continuous simulation model, which can then be applied to generate stream flow patterns. In most cases, a simplified event-based modeling approach using SCS or USACE methods must be relied on to generate the surface water component of a water budget. Several methods have been introduced in previous chapters of this document. Application of any rainfall runoff model must be consistent with the water budget analysis time step.

A common approach for estimating surface water inflows is the application of the SCS curve number method as shown by equation 9.2. This method produces a total volume inflow based on the precipitation, initial abstraction, and land cover as represented in the maximum potential retention variable.

$$Q = \frac{(P - I_a)^2}{(P + S - I_a)} \tag{9.2}$$

where,

Q = runoff depth, mm (in)
P = rainfall depth, mm (in)
I_a = initial abstraction, mm (in)
S = maximum potential retention, mm (in)

Since this approach is based on a 24-h precipitation, it should be applied to a time series of daily rainfall data to produce a corresponding time series of daily runoff data. Two potential limitations to this approach are apparent. First, it assumes that runoff for each day is independent of every other day. For example, if a precipitation event begins at 10 pm and continues until 4 am the next morning, the rainfall and runoff from 10 pm to midnight will be treated independently from that occurring between midnight and 4 am. In such a case, the runoff may be underestimated.

Second, the SCS methodology will not show any runoff for any storm in which initial abstraction is greater than or equal to precipitation. Although this may be an appropriately intuitive result, it should be recalled that the SCS method was developed to generate runoff volumes for relatively large events, not for long-term wetland water budgets. As such, it may underestimate the total volume of runoff throughout a year.

A second approach for generating surface water inflows is to use a continuous simulation model such as EPA's Stormwater Management Model (SWMM). SWMM addresses the limitations of the SCS methodology by performing a continuous accounting of infiltration, evaporation, and runoff. Starting with hourly precipitation data, the model will generate hourly runoff values that take into account antecedent conditions rather than assuming that each computation is independent of the previous computation.

Groundwater is the most challenging of the inflows to estimate. Typically, field measurements taken from monitoring wells over a one or two year period are the only site-specific basis for estimating groundwater availability at a site. These data must be interpreted in the context of whether the data have been collected during a typical period or an atypically dry or wet period. Collected data are then assessed in that context. Knowing whether the groundwater comes from local versus regional aquifers and confined versus unconfined aquifers is also essential to estimating the long-term water availability. Darcy's Law, as shown in Equation 9.3, is a useful way for estimating groundwater flow to a wetland.

$$q = K A \left(\frac{dh}{dx}\right) \tag{9.3}$$

where,

q = discharge, m^3/s (ft^3/s)
K = hydraulic conductivity, m/s (ft/s)
A = cross-sectional area orthogonal to flow, m^2 (ft^2)
$\frac{dh}{dx}$ = hydraulic gradient, m/m (ft/ft)

9.1.6.2 Water Budget Outflows

Outflows primarily include evapotranspiration, surface water outflow, and groundwater outflow (infiltration). One or more of these outflows may be small compared to the others. However, to be conservative in estimating water availability, each outflow is estimated in some manner to avoid overestimating the amount of water available in a wetland.

Evapotranspiration (ET) is the combined effect of water surface evaporation and vegetative transpiration. However, vegetation reduces ET rates to 30 to 90 percent of the rates in open water. That is, the ET from a wetland would generally be less than evaporation from a lake in the same location. In some locations, ET data may be available from state climatological centers or be estimated from pan evaporation rates. If site-specific data are not available, there are several methods for estimating ET. Energy balance methods, such as Penman-Monteith, are very complex and may require data unavailable for most sites. Climatological methods, such as Blaney-Criddle and Thornthwaite-Mather, rely on more commonly available climate-related variables such as solar radiation, temperature, wind speed, and relative humidity.

The Thornthwaite-Mather method requires only monthly mean air temperature and latitude. Equation 9.4 provides monthly potential evapotranspiration. (The equation is presented only in SI units to avoid confusion.)

$$ET_j = 16\left(\frac{10\,T_j}{I}\right)^a \qquad (9.4)$$

where,

ET_j = potential ET in month j, mm
T_j = mean air temperature in month j, (^{o}C)
I = = monthly heat index
a = exponent which is function of I

The monthly heat index and exponent, a, are defined in Equations 9.5 and 9.6, respectively:

$$I = \sum_{j=1}^{12}\left(\frac{T_j}{5}\right)^{1.5} \qquad (9.5)$$

$$a = 0.49 + 0.01791\,I - 0.0000771\,I^2 + 0.000000675\,I^3 \qquad (9.6)$$

The monthly heat index is a function of air temperature computed over a 12-month period. The result of equations 9.4 through 9.6 is a monthly series of potential evapotranspiration values at the Equator (0 degrees latitude). Dunne and Leopold (1978) have developed multiplicative adjustment factors for other latitudes, which are summarized in Table 9.2.

Table 9.2. Thornthwaite-Mather Latitude Adjustment Factors

Latitude	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60 N	0.54	0.67	0.97	1.19	1.33	1.56	1.55	1.33	1.07	0.84	0.58	0.48
50 N	0.71	0.84	0.98	1.14	1.28	1.36	1.33	1.21	1.06	0.90	0.76	0.68
40 N	0.80	0.89	0.99	1.10	1.20	1.25	1.23	1.15	1.04	0.93	0.83	0.78
30 N	0.87	0.93	1.00	1.07	1.14	1.17	1.16	1.11	1.03	0.96	0.89	0.85
20 N	0.92	0.96	1.00	1.05	1.09	1.11	1.10	1.07	1.02	0.98	0.97	0.96
10 N	0.97	0.98	1.00	1.03	1.05	1.06	1.05	1.04	1.02	0.99	0.97	0.96
0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

A second type of outflow is surface water outflow. Surface water outflow is dependent on the storage available in the wetland and the nature of the control structure. If a control structure is present, the elevation and configuration (weirs and orifices) will determine the outflow based on the water surface elevation in the wetland. For many water budgets, all water volume exceeding some control elevation will leave the wetland as surface outflow during a given time step. Only in cases where a very short time step is used will this not be the case.

Groundwater outflow represents the final departure route for water from a wetland. Like groundwater inflow, this route may be quantified by use of Darcy's equation (Equation 9.3). However, the concept of hydraulic gradient, *dh/dx*, is difficult to conceptualize in the vertical direction. For this reason groundwater outflow is conceptualized as infiltration and the quantity *K(dh/dx)* in Darcy's equation is estimated as the infiltration rate for the soils underlying the wetland. Using the area of the wetland as the area in Darcy's equation provides a means for estimating groundwater outflow.

9.1.6.3 Storage

The final component of a water budget is the water storage provided by the wetland. Similar to stormwater management ponds, the storage of a wetland may be described by the use of stage-storage or stage-area curves. Depth in a wetland may range from zero, where there is no surface inundation, to the maximum depth at a control location, for example, at the control structure.

Although, the stage-storage relationship is most useful for water budget computations, the stage-area relationship is useful for determining the extent of inundation, which in turn is needed to determine the wetland areas in which certain types of vegetation may be successful. The water budget will also provide key information on the hydroperiod, which is necessary for selecting appropriate plantings.

9.1.7 Water Budget Application Issues

Once a wetland model has been selected and the essential inflows and outflows are identified, several key decisions must be made to implement a useful water budget. Two of the most critical of these decisions are the analysis time step and the definitions of typical and extreme conditions.

Selection of the analysis time step is based on the variability of the water sources and losses, the methodologies used for estimating water availability, data availability, and resources available to perform the water budget. Most water budgets are performed on a monthly or daily basis, but can also be performed on an hourly basis. The decision should be driven by the needs of the wetland scientist in understanding water availability at a site and designing an appropriate landscape and planting plan.

Hydrologic design goals are based on a combination of depth, duration, and frequency requirements and may be further proscribed to what may be expected to occur during the growing season. It may be necessary to provide for a specified number of consecutive days at a prescribed depth to insure wetland survival. Typically, surface water inflows govern selection of a time step. At a minimum, surface water inflows are calculated on a daily basis. The water budget may then be completed for all inflows and outflows on a daily basis or the daily inflows may be aggregated to a monthly time step and used in a water budget based on monthly time steps. Similarly, runoff values may be calculated on an hourly basis and aggregated to daily values for water budget computations on a daily basis. Water budgets are rarely calculated on a frequency less than a day because sufficient data to do so are not available.

The second critical decision pertains to the definition of typical and extreme conditions. This decision relates to the frequency of inundation and long-term survivability of wetland vegetation. Definition of a typical or average year is often based on total annual precipitation, but may also be selected based on numbers of days with measurable precipitation, total precipitation during the growing season, or other parameters. One limitation of using total annual precipitation is that a year with many smaller storms may have a much different effect on a wetland than one with fewer large storms yet they may have the same total rainfall. Therefore, distribution of rainfall throughout the year should be considered in selecting a typical year. Rainfall histograms are a useful tool for investigating rainfall distribution. Even though a typical year is unlikely to be the governing design event, it is useful to consider a typical year as a reference condition to compare with more extreme years. The definition of a typical year should be made in consultation with the wetland scientist, but is less critical than defining the extreme year.

The extreme year is that year during which the wetland site is tested, but survives with sufficient water. As with the typical year, this may be based on total precipitation for the year, number of days of precipitation, or total precipitation during the growing season. The question becomes, how extreme should the extreme year be for design purposes? For example, if the wetland scientist determines that sufficient water should be available in 9 out 10 years, then the extreme year to be selected for design purposes is the one with a 10 percent (0.1) exceedence probability. For long records this may reasonably be accomplished by ranking the available data. For short records, the available data should be fit to an appropriate probability distribution. Although determination of an extreme year is generally only relevant for extremely dry conditions, the same process may be used to identify extremely wet years if excess water is a concern.

A limitation to selection of particular years (typical and extreme) for design purposes is that there may be anomalies to the distribution of rainfall or other parameters that could result in misleading results. An alternative is to perform water budgets for all years of available data using continuous simulation. Although much more computationally intensive, the full record is used to evaluate a proposed wetland. Short periods of record, however, may lack extreme events or contain an uncharacteristically large percentage of extreme events. Care must be taken to evaluate the representativeness of a given record in using continuous simulation.

9.1.8 Example Application

An example water budget computation is provided in this section. This example is used to illustrate the steps involved in performing a water budget. However, it is not a complete analysis because it only addresses the typical year. It is based on creating a new wetland at a mitigation site upstream of a secondary road crossing of Clear Creek in South Carolina and is adapted from the example provided in the Model Drainage Manual (AASHTO, 2000). It is presented only in SI units to avoid confusion.

The drainage area to the site is 695 ha with an average spring fed baseflow of 0.0005 m^3/s. For purposes of determining evapotranspiration the site is located at latitude of 34 degrees north. Soil permeability, K, is 8×10^{-5} mm/s. The wetland mitigation site is to cover 2.5 ha. The objective is to calculate a monthly water balance for the typical rainfall year. The following steps are applied:

1. Select wetland model.
2. Determine design conditions.
3. Determine inflows and outflows.
4. Obtain data.
5. Analyze Inflows.
6. Analyze outflows.
7. Characterize storage.
8. Calculate water budget.

Selecting the appropriate wetland model for the project is essential for identifying the important components for the water budget. For this site, it is anticipated that the wetland will be created within and adjacent to Clear Creek with surface water being the primary water source. Therefore, the inline stream wetland creation model is appropriate for the site. To be successful, though, it must be verified that Clear Creek is a relatively low energy stream and relatively free of debris.

To create a wetland using the instream model a control structure is required. The dimensions of the control structure are site-specific and depend on the type of vegetation to be established. Initially, it is assumed that the control structure will be designed to create a maximum depth of 1.0 m. If Clear Creek is not a low energy stream free of debris, the control structure will be threatened with erosion and the chances of success with the mitigation design are reduced.

The second step is to determine the design conditions in coordination with the wetlands specialist. Specifically, the hydroperiod (depth, duration, and frequency) goals are established, including definition of typical and extreme conditions. For this example, the wetland scientist has determined that two types of wetland vegetation are desired for this mitigation project: submergents and emergents. This leads to the following two design requirements:

1. Submergents: Provide at least 500 mm of depth over a 0.5 ha area for 90 days in 9 out of 10 years.
2. Emergents: Provide inundation between 0 and 500 mm over 2.0 ha for 90 days in 9 out of 10 years.

The definition of the extreme year can be taken from the design goals where sufficient water is to be provided in 9 of 10 years. Therefore, the extreme dry year is defined as having a 0.10 exceedence probability. Based on the wetland model and the hydroperiod goals, the hydrologic engineer then chooses the computational time step. For this example, a monthly time step is chosen, but surface runoff is calculated on a daily basis.

Having clearly defined design goals is necessary, but not sufficient, for a successful wetland mitigation project. The design process may be iterative. Initial expectations of the type of wetland may not be supported by the water budget. This may alter the wetland design including the hydroperiod goals and perhaps even the wetland model. The water budget analysis may also demonstrate that a wetland of any kind is not viable at a given site.

The third step is to determine the essential inflows and outflows for this particular site based on the wetland model and anticipated data availability. It is conservative and appropriate (except in circumstances where too much water could be a problem) to assume one or more of the inflows to be negligible while focusing on the primary inflow source. For this example, direct precipitation and surface water inflows will be estimated and groundwater inflow will be assumed to be negligible and ignored. For the outflows, evapotranspiration, surface water outflows, and groundwater outflows will be considered.

Once the wetland model and design goals are established, the next step is to obtain the necessary data for the water budget. The wetland model and primary inflows and outflows will determine the emphasis to be placed on certain types of data, but generally data describing soils, topography, and land use/land cover are required to establish drainage area, infiltration parameters, and runoff characteristics.

Surface water contributions can either be established based on gaged streamflow data, if available, or through rainfall/runoff modeling based on precipitation data. At this site there is no gaged streamflow data so rainfall/runoff modeling will be conducted. A representative rainfall gage nearby includes a 47-year period of record covering the years 1949 through 1995. The typical rainfall year for this analysis is chosen to be the median total precipitation year, which is 1968 with 1236 mm of precipitation. Table 9.3 summarizes the daily precipitation in 1968. The driest year on record was 1954 with 654 mm and the wettest was 1964 with 2043 mm. All precipitation at this site was recorded as rainfall.

Evapotranspiration may be estimated based on climatological data including mean monthly temperature. For this example, evapotranspiration will be based on the Thornthwaite-Mather methodology, which requires only site latitude and monthly temperatures. Average monthly temperatures for the site are presented in Table 9.4.

Table 9.3. Daily Precipitation (mm) for 1968 (Typical Year)

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	12.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.0	0.0	0.0	0.0
2	2.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.0
3	2.5	0.0	0.0	11.4	2.3	5.6	0.0	0.0	0.0	0.0	0.0	1.3
4	6.6	0.0	0.0	0.0	0.0	0.0	54.9	0.0	0.0	0.0	0.0	14.5
5	0.0	0.0	0.0	14.5	2.3	0.0	37.1	0.0	0.0	0.0	0.0	2.8
6	7.1	0.0	0.0	0.0	0.8	0.0	24.9	0.5	42.9	24.1	0.0	0.0
7	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.5	0.0	0.0
8	0.0	0.0	0.0	0.0	0.0	31.8	0.0	0.0	0.0	3.8	0.0	0.0
9	4.1	0.0	0.0	16.3	0.0	10.9	0.0	0.0	0.0	0.0	30.2	0.0
10	70.9	0.0	0.0	2.0	0.0	38.9	38.4	0.0	3.0	0.0	1.5	0.0
11	0.5	0.0	16.5	0.0	0.0	1.0	21.1	0.0	0.0	0.0	38.1	0.0
12	18.5	0.0	6.4	0.0	3.0	0.0	7.1	6.1	0.0	0.0	2.3	0.0
13	5.1	0.0	6.6	0.0	2.0	29.7	13.0	1.5	0.0	0.0	0.0	0.0
14	0.0	0.0	0.0	0.0	52.8	0.0	0.0	2.5	0.0	0.0	0.0	0.0
15	0.0	0.0	0.0	4.3	11.4	0.0	0.0	0.0	0.0	0.0	0.0	4.3
16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.8	3.3	0.0
17	0.0	0.0	8.6	0.0	6.6	0.0	0.0	0.0	0.0	7.1	0.0	0.0
18	0.0	0.0	0.3	0.0	4.8	4.1	0.0	0.0	0.0	3.0	6.4	0.0
19	0.0	0.0	0.0	0.0	5.3	1.0	26.4	0.0	0.0	66.0	0.0	0.0
20	0.0	0.0	0.0	0.0	0.0	0.0	12.2	4.8	0.0	50.5	0.0	0.0
21	0.0	1.3	0.0	0.0	0.0	0.0	0.0	0.8	0.0	0.0	0.0	0.0
22	0.0	1.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	0.0	0.0	0.8	3.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.7
24	20.1	0.5	4.1	8.9	0.0	4.3	0.0	0.0	0.0	0.0	0.0	7.6
25	0.3	0.0	0.0	0.0	0.0	10.2	0.0	2.0	0.0	0.0	0.0	0.0
26	0.0	0.0	0.0	0.0	1.0	0.0	0.0	4.3	0.0	0.0	0.0	0.0
27	0.0	0.0	0.0	1.0	11.2	0.0	0.0	0.0	11.9	0.0	0.0	0.0
28	0.0	0.5	0.0	5.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.0	20.3	0.0	47.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	13.2
30	0.0		5.1	0.0	2.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
31	0.0		0.5		0.0		0.8	5.6		0.0		12.4
Total	150.9	29.0	48.9	114.8	105.8	137.5	235.9	28.1	60.8	159.8	81.8	82.8

Table 9.4. Monthly Average Temperatures for 1968 (Typical Year)

Month	Mean Temp (°C)
January	5.1
February	4.5
March	12.7
April	17.7
May	20.9
June	25.5
July	26.9
August	28.3
September	22.4
October	17.8
November	12.0
December	5.9

Once data collection has been completed, the inflows to the wetland site are computed. In this example, they are direct precipitation and surface inflow. The largest of these in this example is the surface water inflow associated with rainfall runoff.

The SCS method is applied assuming average antecedent moisture conditions and that each day of rainfall generates a separate runoff event. Based on the land uses and soil types in the contributing watershed a curve number (CN) of 64 is estimated. Maximum potential retention, S, is equal to 25.4[(1000/CN)-10] = 143 mm. Assuming that initial abstraction, I_a, is 20 percent of maximum potential retention, then I_a = 0.2(143) = 28.6 mm. Therefore, only days with precipitation greater than 28.6 mm will generate runoff. Table 9.5 summarizes the runoff computations for the 14 days in 1968 generating runoff. For the monthly water budget, the daily values within a month are added together to estimate the monthly runoff.

Table 9.5. Runoff Computations for 1968

Month	Day	Daily Precipitation (mm)	Q (mm)	Volume (m^3)	Total Runoff Volume per Month (m^3)
January	10	70.9	9.67	67226	67226
April	29	47.5	2.21	15384	15384
May	14	52.8	3.51	24408	24408
June	8	31.8	0.07	495	5392
June	10	38.9	0.70	4836	
June	13	29.7	0.01	61	
July	4	54.9	4.10	28466	36195
July	5	37.1	0.48	3336	
July	10	38.4	0.63	4394	
September	6	42.9	1.31	9072	9072
October	19	66.0	7.77	53990	74262
October	20	50.5	2.92	20272	
November	9	30.2	0.02	127	4264
November	11	38.1	0.60	4137	

Base flow is estimated in this example as a constant value equal to 0.0005 m^3/s, which on a monthly basis equals 1314 m^3/month. Base flow should be based on site-specific measurements, if available. Although a constant base flow assumption throughout the year has been made in this case, such an assumption would likely be inappropriate if base flow is a significant part of the budget and significant variations are known to occur throughout the year. In this example, base flow is a minor component of the water budget.

Step 6 is to analyze the outflows. First, evapotranspiration is estimated using the Thornthwaite-Mather approach based on mean monthly temperature and latitude. Using the monthly average temperatures for 1968 yields ET computations summarized in Table 9.6. Volumetric ET outflows will be dependent on the surface area of the wetland, which may also vary throughout the year.

Table 9.6. ET for 1968

Month	Mean Temp (^oC) T_i	$(T_i/5)^{1.5}$	ET (mm/month)	Correction Factor	Corrected ET (mm/month)
January	5.1	1.03	7.2	0.84	6.1
February	4.5	0.85	5.8	0.91	5.3
March	12.7	4.05	36.4	1.00	36.4
April	17.7	6.66	65.6	1.08	70.8
May	20.9	8.55	88.0	1.16	102.1
June	25.5	11.52	125.1	1.20	150.2
July	26.9	12.48	137.6	1.19	163.7
August	28.3	13.47	150.5	1.13	170.0
September	22.4	9.48	99.5	1.03	102.5
October	17.8	6.72	66.2	0.95	62.9
November	12.0	3.72	32.9	0.87	28.7
December	5.9	1.28	9.4	0.82	7.7
	I =	79.80			
	a =	1.77			

Surface water outflow is determined by the control structure, which is designed to maintain a maximum depth of 1 m. Therefore, all excess inflows stored at a depth greater than 1 m will be assumed to flow over the control structure as outflows. For short computational time steps, it may be necessary to compute outflows in a situation such as this using the weir equation as part of a storage routing procedure in the wetland. However, in this case, the outflows are rapid compared to the monthly time step and it is assumed that by the end of the month excess inflows will have been released from the control structure.

Finally, groundwater outflow (infiltration) will be based on Darcy's equation with an infiltration rate of $8x10^{-5}$ mm/s (0.210 m/month).

The next step is to characterize the storage characteristics of the wetland. The proposed grading on this site results in the stage-storage and stage-area curves shown in Figures 9.8 and 9.9.

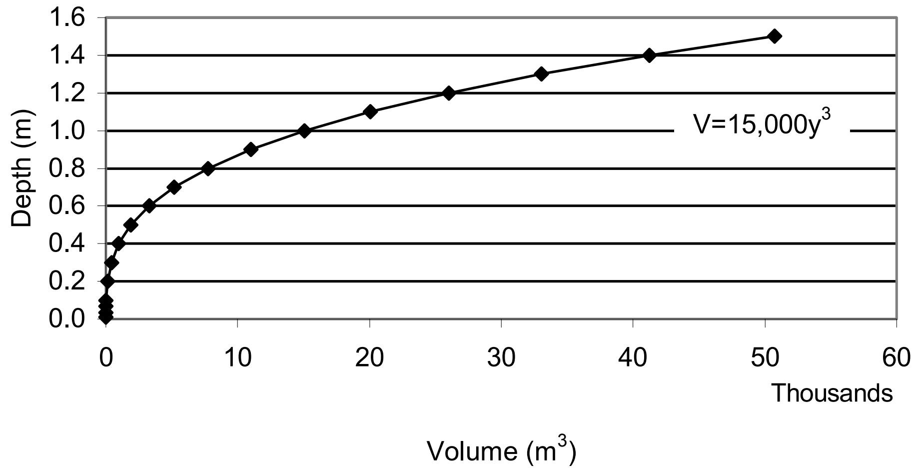


Figure 9.8. Stage-storage curve for proposed wetland

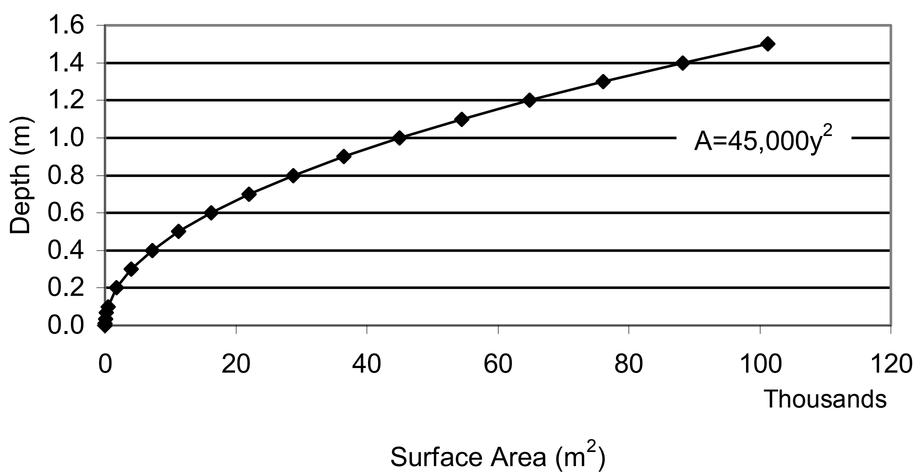


Figure 9.9. Stage-area curve for proposed wetland

The eighth and final step is to combine all of the components and compute the water budget. This process is summarized in Table 9.7, which shows the water budget over the entire year. The first row in the table establishes the starting conditions for the analysis. In this case, the starting conditions are estimated to be the depth and total water volume stored in the wetland and the end of December 1967. The first column designates the month. The next two columns relate to the surface water inflows of direct runoff and base flow that have been estimated in volumetric terms. For example in April of 1968, the direct runoff of 15,384 m^3 is combined with the 1314 m^3 of base flow. When these are added to the total volume in the wetland at the end of the prior month (4800 m^3) the interim volume estimate is 21,498 m^3. Inspection of the stage-storage curve reveals that the interim depth corresponding to this volume is 1.13 m after considering all inflows except direct precipitation. Deducting the outflows resulting from potential evapotranspiration (PET) of 0.07 m and infiltration of 0.21 m and adding the direct precipitation of 0.115 m yields a revised depth in the wetland of 0.96 m. Since this is less than the 1.0 m control depth, there is no surface water outflow for the month of April. This result is contrasted with the month of January where the depth after outflows is 1.65 m. Depth at the end of the month in January is then adjusted to 1.0 m with the difference attributed to surface water outflow.

Returning to the April computation, a depth of 0.96 m has been determined for the end of the month. Consulting the stage-storage curve, a volume of 13,326 m^3 is determined as the month-ending storage volume. This value is used as the starting point for the calculations for the subsequent month.

Table 9.7. Monthly 1968 Water Budget

	Runoff Volume (m^3)	Base Flow (m^3)	Total Volume (m^3)	Depth after inflows (m)	PET (m)	Infiltration (m)	Precip-itation (m)	Depth after outflows (m)	Depth at end of month (m)	Total Volume (m^3)
Dec-67									0.80	7680
Jan-68	67226	1314	76220	1.72	0.01	0.21	0.151	1.65	1.00	15000
Feb-68	0	1314	16314	1.03	0.01	0.21	0.029	0.84	0.84	8957
Mar-68	0	1314	10271	0.88	0.04	0.21	0.049	0.68	0.68	4800
Apr-68	15384	1314	21498	1.13	0.07	0.21	0.115	0.96	0.96	13326
May-68	24408	1314	39048	1.38	0.10	0.21	0.106	1.17	1.00	15000
Jun-68	5392	1314	21706	1.13	0.15	0.21	0.138	0.91	0.91	11240
Jul-68	36195	1314	48749	1.48	0.16	0.21	0.236	1.34	1.00	15000
Aug-68	0	1314	16314	1.03	0.17	0.21	0.028	0.68	0.68	4642
Sep-68	9072	1314	15029	1.00	0.10	0.21	0.061	0.75	0.75	6302
Oct-68	74262	1314	81879	1.76	0.06	0.21	0.160	1.65	1.00	15000
Nov-68	4264	1314	20578	1.11	0.03	0.21	0.082	0.95	0.95	13031
Dec-68	0	1314	14345	0.99	0.01	0.21	0.083	0.85	0.85	9223
Total	236205	15768			0.91	2.52	1.236			

A clearer picture of the water budget is obtained by graphing the maximum end of month values as shown in Figure 9.10. Note that the depth is not allowed to exceed 1.0 m. Depending on the growing season, it appears that there is sufficient water available for the wetland during the typical year.

At this point, it is appropriate to consider the sensitivity of these computations to the assumed starting conditions. In this case, there are significant runoff inflows in January so the results are expected to be insensitive to relatively wide fluctuations in starting conditions.

Figure 9.11 makes use of the stage-area curve to display extent of inundation during each month. This presentation shows that at the end of March and August the inundated surface area is a low of 20,000 m^2, while at the end of several other months the surface area is more than doubled. Maximum surface area as determined by the control structure is 45,000 m^2.

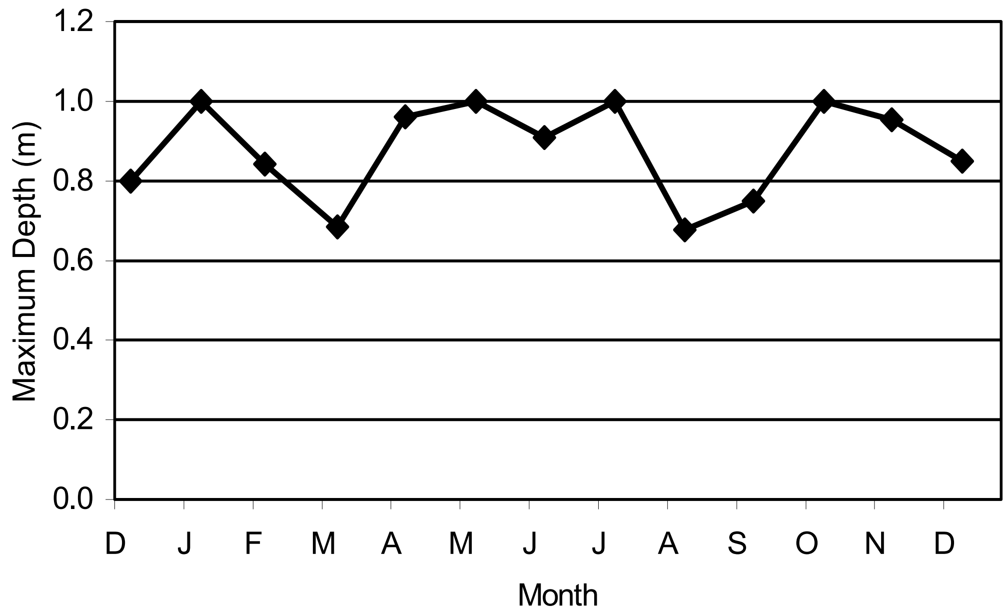


Figure 9.10. Monthly 1968 Water Budget

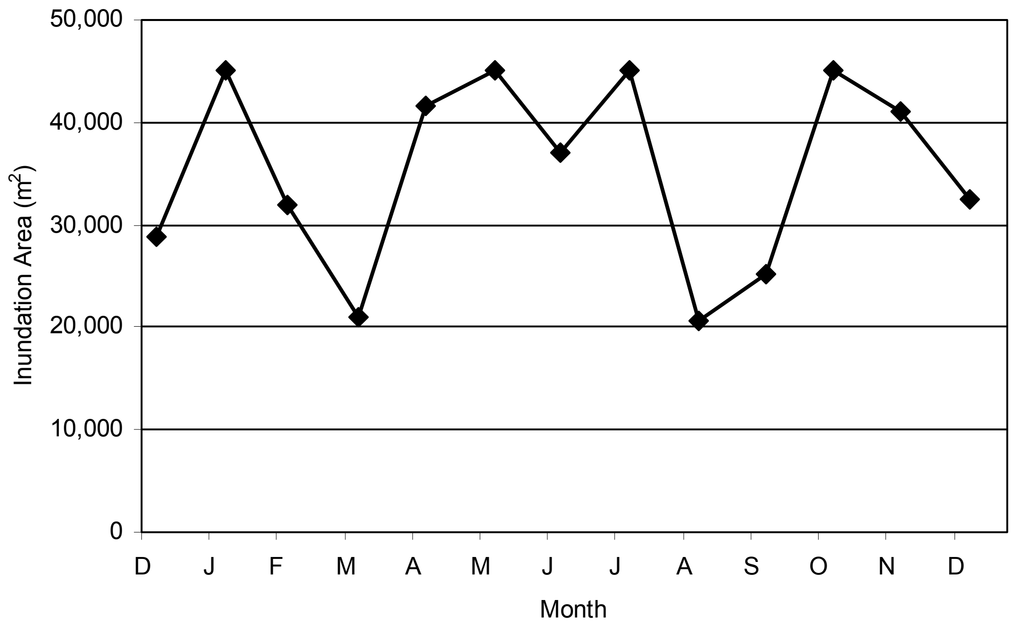


Figure 9.11. Inundation area for monthly 1968 water budget

Recall the design requirements for this example:

1. Submergents: Provide at least 500 mm of depth over a 0.5 ha area for 90 days in 9 out of 10 years.
2. Emergents: Provide inundation between 0 and 500 mm over 2.0 ha for 90 days in 9 out of 10 years.

Have these requirements been met? It would appear that they have for the typical year (1968), but the data need to be presented so as to allow a more conducive assessment of design requirements. To accomplish this, the month end depth values are interpolated to estimate daily depth values. These daily values are then ordered from highest to lowest and plotted as shown in Figure 9.12. From this depth-duration curve, it can be determined what depths are being experienced over which durations. Reading off of the curve for a 90-day duration yields a maximum depth of 0.97 m, meaning that the wetland is at least 0.97 m deep at the control structure for 90 days.

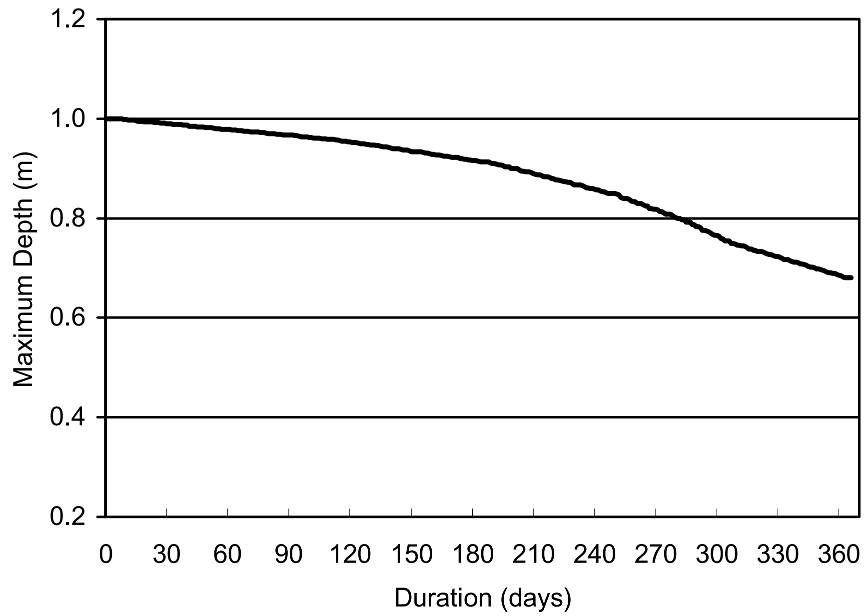


Figure 9.12. Depth-duration curve for 1968 monthly water budget

This result must then be translated to surface area to assess how much inundation occurs at certain depths throughout the wetland. Returning to the stage-area curve, it is determined that at a maximum depth of 0.97 m a total of 4.2 ha are inundated for a duration of 90 days. These depths range from 0 to the maximum of 0.97 m. Subtracting 0.5 m from the 0.97 m it is further determined that 1.0 ha of area is inundated to a minimum depth of 500 mm with the remaining 3.2 ha inundated to depths between 0 and 500 mm. From this it is concluded that the minimum limits of 0.5 and 2.0 ha, respectively, are satisfied for the typical year.

It is still necessary to determine if this requirement is satisfied in 9 out of 10 years. Two strategies may be pursued. One is to identify the year within the period of record that best represents the 0.1 exceedence probability year and perform a water budget computation for that year. The second alternative is to perform water budget computations for all years and assess whether the requirements are met for 42 (90%) of the 47 years.

9.1.9 Sensitivity Analysis

Water budgets rely on a number of engineering judgments to properly implement. Sensitivity analysis increases the chances of successful design by exploring the potential changes in conclusions that may result from changes in assumptions and may also identify data collection needs required to reduce uncertainty in the conclusions. In general, sensitivity analysis enables preparation of more robust water budgets on which to base design recommendations.

For example, if it is initially assumed that that base flow is not a significant contributor to the viability of a proposed wetland mitigation project, a constant base flow may be a reasonable assumption. A sensitivity analysis on base flow might be conducted where base flow is reduced by 50% and increased by 100%. If these changes do not change the results of the water budget analysis, then a greater degree of confidence in the conclusions is warranted. If, however, these scenarios result in changed conclusions, more attention and/or data pertaining to base flow is indicated.

Another area of sensitivity analysis that should be addressed prior to completing a water budget is the computational time step. In the example water budget presented in the previous section, the conclusion was drawn that the inundation areas suitable for submergents and emergents were 1.0 and 3.2 ha, respectively, in the typical year. Performing a water budget analysis on the same year using a daily time step yields a somewhat different result as is shown in Figure 9.13. Creating an analogous depth-duration curve and applying the same procedures illustrated in the example leads to the result that areas suitable for submergents and emergents are 0.8 and 3.1 ha, respectively. Based on this result, the monthly time step overestimated the available area compared with the daily time step. For the typical year, design criteria are still attained, but this may not be the case in drier years. When data are available to support the analysis, a daily time step is preferable to a monthly time step.

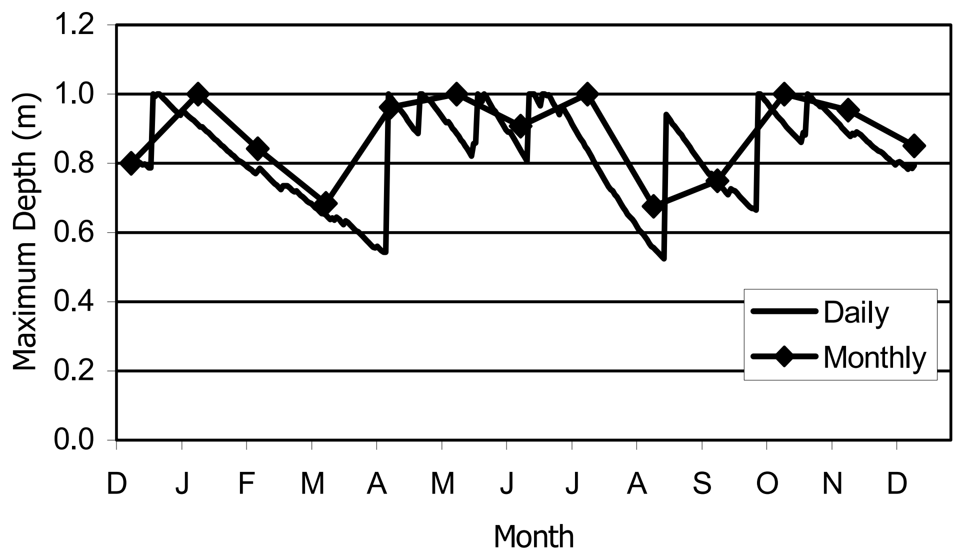


Figure 9.13. Comparison of monthly and daily water budgets

9.2 SNOWMELT

Mountainous regions of the United States may derive up to 90 percent of the their annual water supply from snowmelt. Snowmelt is also an important source of water in the Great Lakes region and Northeastern United States. Snowmelt can play an important role in the annual stream flow variation and can also cause flood damage to roads or contribute to flood hydrographs. To fully understand the full range of flood sources from a watershed, the hydrologist should understand the snowmelt process. Figure 9.14 shows the variation of annual snowfall in the United States.

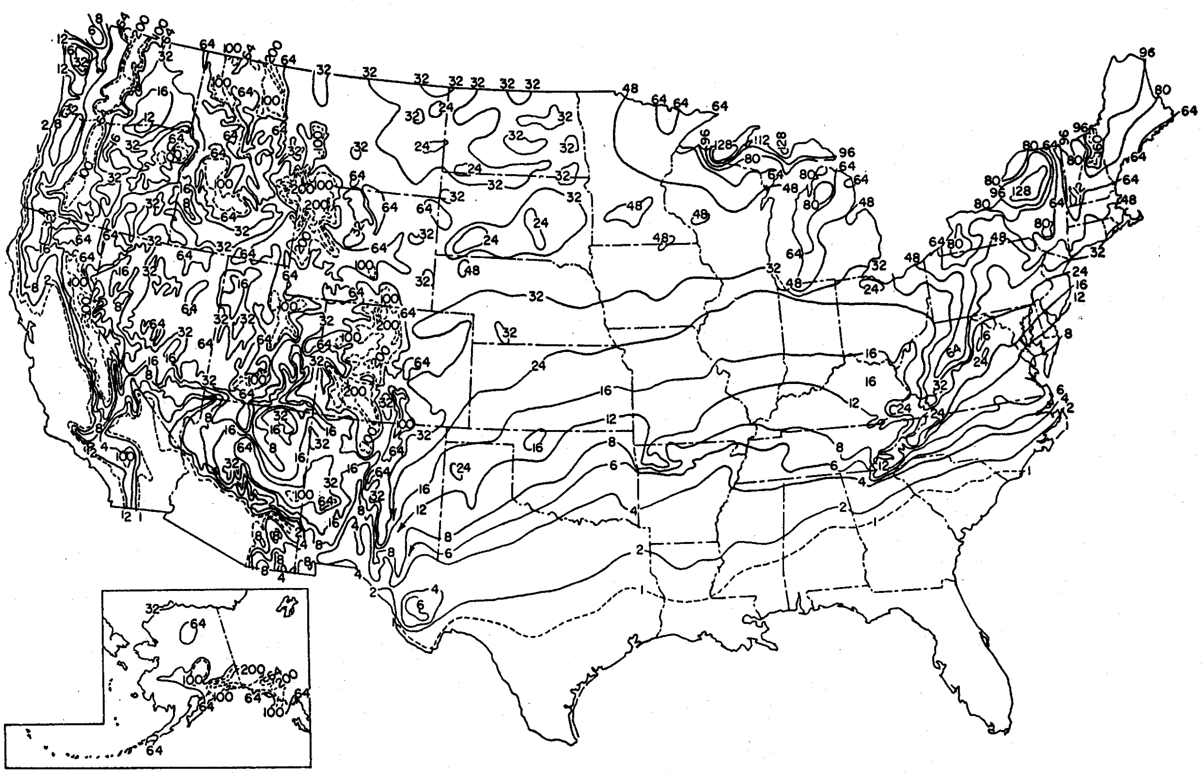


Figure 9.14. Mean Annual Snowfall in the United States in Inches (1931 to 1952).
(U.S. Weather Bureau, 1960)

9.2.1 Fundamental Properties of Water, Snow, and Ice

To fully understand the processes affecting the relationship between water, snow, and ice, the terms used to describe these relationships should be defined.

Latent heat is the amount of heat needed to change the phase of a compound with no change in temperature. *Specific heat* is the amount of heat necessary to raise the temperature of a compound over a given temperature interval without a change in state. *Fusion* is the phase conversion of a solid to a liquid. *Vaporization* is the phase conversion of a liquid to a gas, and *sublimation* is the phase conversion of a solid to a gas. The *latent heat of fusion* of water is the heat necessary to change ice to water and is 334.9 kJ/kg. The *latent heat of vaporization* of water at 0^{o}C is 596 calories per gram, and the *latent heat of sublimation* is 2830 kJ/kg.

Vapor pressure is the pressure that would be exerted by water vapor if all other gases were absent. Molecule movement follows the gradient of vapor pressure. The rate at which water molecules leave a water surface is dependent on the temperature. The *saturation vapor pressure* occurs when an equilibrium condition occurs and the vapor pressure of the liquid is equal to the partial pressure of the molecules from the liquid in the gas above the liquid surface and the amount of molecules entering the liquid is equal to the amount of molecules leaving the liquid.

Evaporation occurs when more water molecules leave a water surface than enter it. The evaporation rate is dependent on the vapor pressure gradient at the water surface. The vapor pressure gradient is in turn dependant on the temperature of the air and water, wind velocity, atmospheric pressure, shape and nature of water surface, and water quality. *Condensation* occurs when more water molecules enter a water surface than leave it.

Relative humidity is the ratio of the actual vapor pressure to the saturation vapor pressure at a given temperature. The *dew point temperature* is the temperature to which the air must be cooled to reach the saturation vapor pressure at a constant pressure and water-vapor content.

Because the density of newly fallen snow can vary greatly, the amount of water that can be obtained from the snow will vary. *Snow Water Equivalent* (SWE) is the depth of water that is obtained by melting the snow from a given snow event and it usually expressed in units of equivalent depth of water. New snow normally has a water content of about 10 percent, which would have a water equivalent of 1 mm for a 10 mm snowfall, but it may vary from 5 percent to 25 percent. *Density* is the percentage of snow volume that would be occupied by its water equivalent. The density of fallen snow generally increases at a constant rate over time and is normally at its greatest density just before the snowmelt season begins. Snow may have a density of 60 percent before the snowmelt season begins.

Thermal quality is the ratio of the weight of ice to the total weight of the sample, or alternatively, the ratio of heat required to melt a unit mass of snow to that of ice at 0^oC (23^oF). Typically, thermal quality is about 0.95, but during periods of rapid melt, it may drop to 0.7 or less.

Cold content, or heat deficit, is the amount of energy required to raise the temperature of the snowpack to 0^oC (32^oF).

9.2.2 Snowmelt Runoff

When the air temperature begins to warm, the snowpack begins to melt, beginning first at the surface. At first the melted water only moves slightly below the surface, where it freezes again when it contacts the colder snow lying beneath. The snow pack is slowly heated from the heat of fusion as the melted water is refrozen. Heat is also available from the overlying air and from the underlying ground. As the temperature of the snowpack rises, the melted water flows deeper and deeper into the pack. The melted water is held on the snow or ice crystals in capillary films until the liquid water holding capacity of the snow is reached. At this point the snow is said to be *ripe* and any further melting will result in runoff.

The snowmelt process converts the snowpack to water. The rate at which this conversion occurs varies widely due to a number of variables. The rate of snowmelt can be estimated with the following equation:

$$M = \frac{E_m}{L \rho_w B} \qquad (9.6)$$

where,

M = snowmelt runoff rate (m/day)
E_m = energy flux for melt (kJ/m^{2}day)
L = latent heat of fusion for ice = 334.9 kJ/kg
ρ_w = density of water = 1000 kg/m^3
B = thermal quality of snow (dimensionless)

The thermal quality of the snow is related to the water content of the snow. A fully ripe snowpack normally contains about 3 percent to 5 percent of liquid water, so the thermal quality of the snowpack would range from 0.95 to 0.97.

The energy flux is estimated by summing the various components that add energy to and take energy away from the snowpack, which is shown in the following equation. Most analyses assume that the energy inputs must satisfy the cold content before melting can begin.

$$E_m = E_{sn} + E_{\ln} + E_h + E_e + E_p + E_g - \Delta E_i \qquad (9.7)$$

where,

E_{sn} = energy from net shortwave (solar) radiation
E_{ln} = energy from net longwave radiation
E_h = energy from convective heat exchange
E_e = energy from latent heat of condensation
E_p = energy from heat convected by precipitation
E_g = energy from heat conducted by ground
ΔE_i = change in internal energy storage (cold content)

The method to obtain estimates for each of these energy subcomponents will be discussed in more detail in the following sections.

9.2.3 Snowmelt Processes

Factors which cause the snowmelt to occur include radiation, air convection, vapor condensation, warm rain (advection), and ground conduction. Air convection, vapor condensation, and radiation are usually the most important variables affecting snowmelt and runoff, but rainfall can sometimes have a significant effect on peak flows. Usually the effect of ground conduction is negligible.

9.2.3.1 Radiation

Solar energy is a source of energy that contributes to melting the snowpack. The intensity of solar radiation at the edge of the earth's atmosphere and normal to the path of radiation is a nearly constant 1.35 kJ/m^2/s. Several factors, such as cloudiness, latitude, season of the year, time of day, topography, snow cover, and vegetative cover, affect the amount of solar radiation that actually reaches the ground. Figure 9.15 supplies an estimate of the solar radiation according to season and latitude (1 langley = 41.9 kJ/m^2). *Albedo* is the reflectivity of shortwave radiation of a snowpack. The albedo for snowpack ranges from about 40 percent for melting late-season snow to 80 percent to 90 percent for freshly fallen snow. The net energy supplied by solar radiation to the snowpack can be estimated from the following equation.

$$E_{sn} = (1 - A) I_i \quad (9.8)$$

where,

A = albedo (decimal fraction)

I_i = incident solar radiation (kJ/m^{2}day)

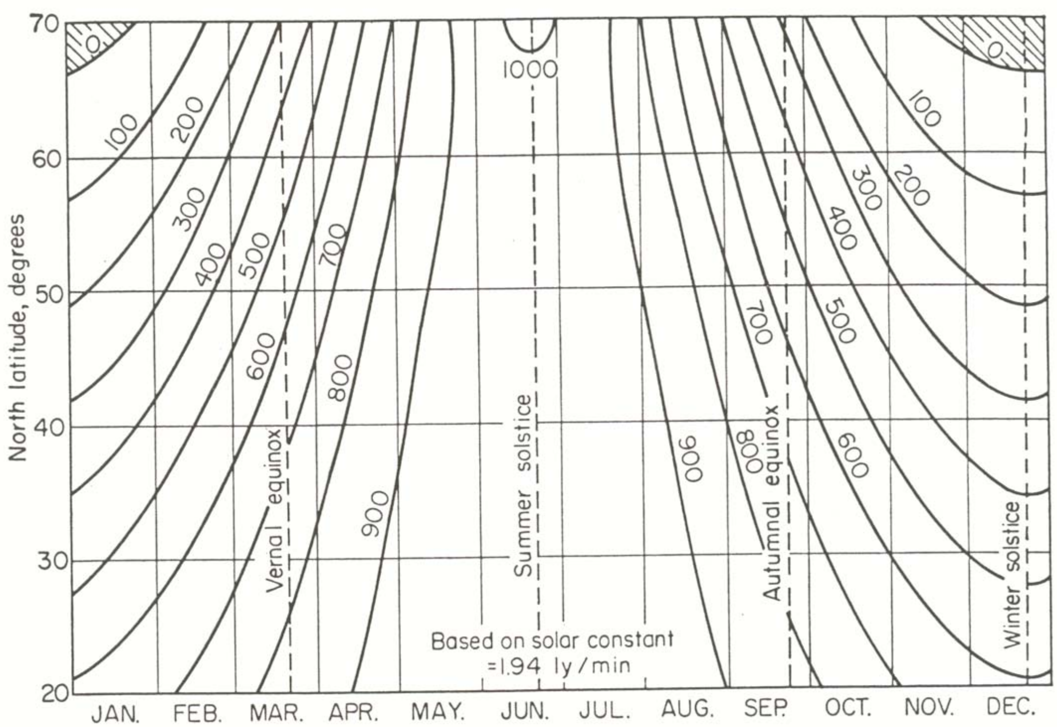


Figure 9.15. Seasonal and Latitudinal Variation of Daily Solar Radiation (langleys)

(U.S. Army Corps of Engineers, 1998)

The portion of shortwave radiation not reflected and available for snowmelt may become longwave radiation. The snowpack loses longwave radiation back to the atmosphere, and the atmosphere (clouds) and trees reflect the longwave radiation back to the snowpack. If the atmosphere has clear skies, the back radiation is generally less that the snowpack loss. If the skies are cloudy or if there is a forest canopy, the back radiation may be greater than the snow pack loss. Snowpacks lose energy through longwave radiation according to the Stefan-Boltzman law for a blackbody. A blackbody absorbs all radiation incident to it and emits radiation according to the Stefan-Boltzman law. The following equation is for longwave radiation emitted to the atmosphere from the snowpack and is not net longwave radiation. It does not account for the radiation reflected back to the snowpack.

$$E_l = \varepsilon \sigma T_s^4 \quad (9.9)$$

where,

E_l = longwave radiation (kJ/m^{2}day)

ε = 0.99 for clean snow

σ = Stefan-Boltzman constant = 5.735x10^{-11} (kJ/m^{2}sK^4)

T_s = snow surface temperature (K)

Figure 9.16 presents the daily snowmelt due to shortwave radiation and net longwave radiation for spring and winter. The figures show that total radiation melt is greater in the spring than in the winter. Spring radiation melt decreases with increasing cloud cover and decreasing cloud height, but winter radiation melt increases with increasing cloud cover and decresing cloud height. This emphasizes that longwave radiation has a more dominant role in the winter.

9.2.3.2 Air Convection

Convection is the process by which heat is transferred from the overlying air to the snow pack. The amount of snowmelt is directly related to wind velocity and air temperature. The energy applied to the snowpack from convective processes can be estimated with the following equation:

$$E_h = D_h u_z (T_a - T_s) \qquad (9.10)$$

where,

D_h = bulk heat transfer coefficient (kJ/m^3 ^{o}C)

u_z = wind speed above snow surface (m/s)

T_a = air temperature (^{o}C)

T_s = snow temperature at surface (^{o}C)

The bulk heat transfer coefficient is a function of atmospheric pressure and must be determined experimentally.

9.2.3.3 Vapor Condensation

When warmer, moisture-laden water is brought into contact with the cooler snow surface by turbulent mixing in the atmosphere, the water vapor condenses to the liquid phase on the snow pack and releases energy. The heat released by the water vapor as it converts to liquid water on the snow becomes available to melt snow. The energy released to the snow is estimated by the following equation:

$$E_e = D_e u_z (e_a - e_s) \qquad (9.11)$$

where,

D_e = bulk latent heat transfer coefficient (kJ/m^3 Pa)

u_z = wind speed above snow surface (m/s)

e_a = atmospheric vapor pressure (Pa)

e_s = vapor pressure at snow surface (Pa)

The dew point temperature must exceed 0^{o}C (32^{o}F) for condensation melt to occur, and if the dew point temperature drops below 0^{o}C (32^{o}F), evaporation occurs at the snow surface.

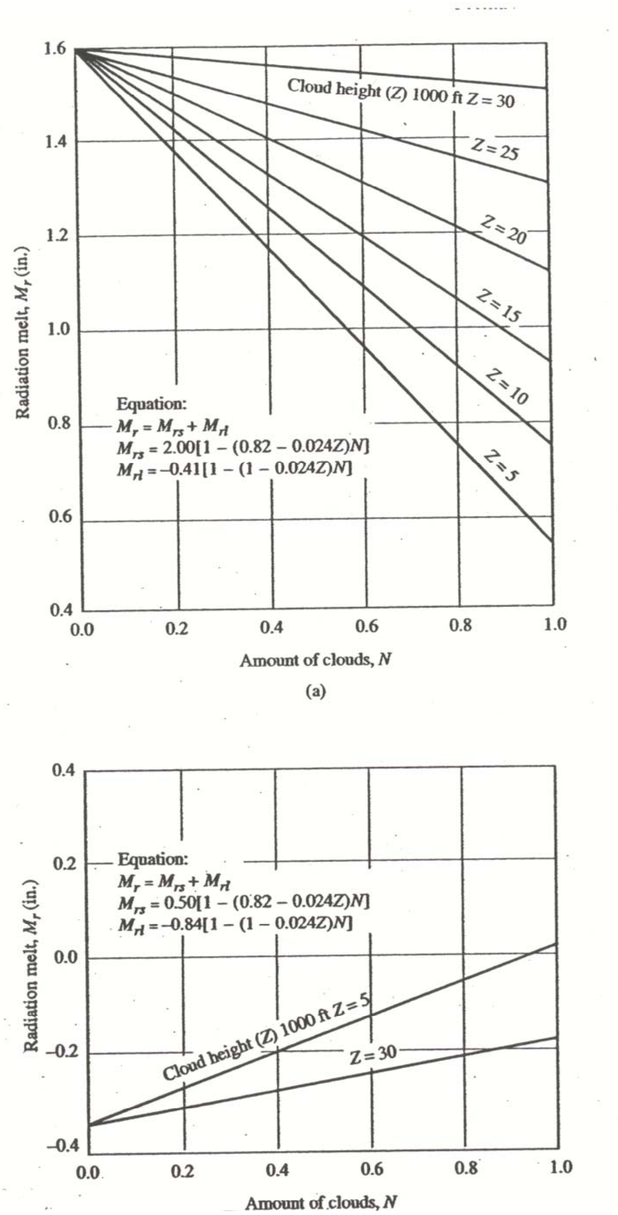


Figure 9.16. Daily Snowmelt due to Shortwave Radiation and Net Longwave Radiation in the Open with Cloudy Skies

(a) During spring , May 20; and (b) During winter, February 15
(U.S. Army Corps of Engineers, 1956)

9.2.3.4 Warm Rain (Advection)

If the temperature of the rainfall is close to freezing, the amount of energy supplied to the snowpack will be small, but if the temperature is higher, then rain drops can be a significant heat source. Generally, the temperature of the rain is assumed to be the air temperature. The following equation provides an estimate of the energy supplied to the snowpack from rain.

$$E_p = C_p \, \rho_w \, P_r \left(T_r - T_s\right) \tag{9.12}$$

where,

C_p = specific heat of rain (kJ/kg ^{o}C)
ρ_w = density of water (kg/m^3)
P_r = rain intensity (m/h)
T_r = rain temperature (^{o}C)
T_s = snow temperature at surface (^{o}C)

9.2.3.5 Ground Conduction

Ground conduction melt occurs when heat from the ground is transferred up to the snowpack. Thermal energy is stored in the ground during the preceding summer and fall. Ground conduction is normally negligible when considering daily melt rates, but it may be significant when used for seasonal melt calculations. The energy from ground conduction can be estimated from the following equation:

$$E_g = k \frac{\Delta T_s}{\Delta z} \tag{9.13}$$

where,

k = thermal conductivity of soil (kJ/m day ^{o}C)
$\frac{\Delta T_s}{\Delta z}$ = temperature gradient from soil to snow (^{o}C/m)

Daily, the melt rate, M, due to ground conduction may vary from 0.25 mm (0.01 in) per day to 0.75 mm (0.03 in) per day. The melt caused by ground conduction provides a constant source of water to the ground, so when other favorable conditions for snowmelt occur, less water is able to infiltrate into the ground and runoff is greater.

9.2.4 Snowmelt Modeling

The various components contributing to snowmelt are estimated using complex, non-linear relationships that require data that is generally unavailable or hard to collect. Several methodologies have been developed that approximate the concepts previously discussed using simplified equations with variables that are generally easy to obtain or find. As with any model, it is important to verify or validate any snowmelt analysis by comparing the results of the model with observed runoff rates.

9.2.4.1 Energy Budget Method

The energy budget method attempts to follow the theoretical relationships previously discussed. The Corps of Engineers has developed approximations of these complex relationships using regression analyses, linearizing the equations, and using representative and easily obtained parameter values (USACE, 1998). The Corps of Engineers has developed a series of equations for rain-free snowmelt events and also for rain-on-snow events. Different equations are also available for varying degrees of forest canopy. Only some of these equations will be presented here. For the full range of equations and for a full discussion of their development, the reader should consult the Corps of Engineers' *Runoff from Snowmelt* (USACE, 1998).

9.2.4.1.1 Rain-on-Snow Snowmelt

The Corps of Engineers has developed two equations to estimate snowmelt when rain is a contributing factor. Each equation applies to a different extent of forested canopy. The following equation for SI units is used in the HEC-1 computer program developed by the Corps of Engineers (USACE, 1998):

$$M = C\left[\left(1.33 + 0.552\,v + 0.0126\,P_r\right)\left(T_a - T_f\right) + 2.3\right] \qquad (9.14\ SI)$$

where,

M = snowmelt runoff depth (mm/day)
C = coefficient (1 in most cases)
v = wind speed 15 meters above snow (m/sec)
P_r = rainfall (mm/day)
T_a = air temperature (^oC)
T_f = temperature at which melt occurs (usually assumed to be 0^oC) (^oC)

The equation using CU units is as follows (USACE, 1998):

$$M = C\left[\left(0.029 + 0.00504\,v + 0.007\,P_r\right)\left(T_a - T_f\right) + 0.09\right] \qquad (9.14\ CU)$$

where,

M = snowmelt runoff depth (in/day)
C = coefficient (1 in most cases)
v = wind speed 50 feet above snow (mi/hour)
P_r = rainfall (in/day)
T_a = air temperature (^oF)
T_f = temperature at which melt occurs (usually assumed to be 32^oF) (^oF)

These equations are valid for conditions where the percent of forest canopy ranges from 10 percent to 80 percent. For forest canopy conditions outside this range, the engineer may want to consider using other equations provided in the Corps of Engineers' *Runoff from Snowmelt* (USACE, 1998).

The coefficient, C, is used to calibrate model results to existing data or to account for conditions that are slightly different than those assumed to develop this model. The first term in the equations accounts for net longwave radiation. The second term combines the effect of convection and condensation on snowmelt, and the third term accounts for the energy contributed by rain. The fourth and final term (a constant) accounts for shortwave radiation and ground melt. Since these equations are for rainy days, it is assumed that there is a full cloud cover.

During a rain event, convection and condensation are the primary method by which heat is introduced to the snowpack causing snowmelt. Because it is assumed that there is a full cloud cover, solar radiation is slight and longwave radiation can be reliably estimated from theoretical considerations. During rain-free periods, shortwave and longwave radiation become significant and convection and condensation are less critical.

9.2.4.1.2 Rain Free Snowmelt

The Corps of Engineers has also developed four equations to estimate snowmelt when rain is not a contributing factor. These rain free equations apply to different ranges of forested canopy. The following equation (SI units), used in the HEC-1 computer program, assumes a forest canopy of 50 percent and is considered valid for a range of forested canopy of 10 percent to 60 percent (USACE, 1998).

$$M = C\left[0.0012\, I_i (1 - A) + 0.66\,(T_a - T_f) + 0.112\, v\,(T_a - T_f) + 0.40\, v\,(T_d - T_f)\right] \quad (9.15\ SI)$$

where,

- M = snowmelt runoff depth (mm/day)
- C = coefficient (1 in most cases)
- v = wind speed 15 meters above snow (m/sec)
- I_i = solar radiation (kJ/m^{2}day)
- A = Albedo (dimensionless)
- T_a = air temperature (^{o}C)
- T_f = temperature at which melt occurs (usually assumed to be 0^{o}C) (^{o}C)
- T_d = dew point temperature (^{o}C)

The equation using CU units is as follows (USACE, 1998):

$$M = C\left[0.0021 I_i (1 - A) + 0.0145(T_a - T_f) + 0.0011\, v\,(T_a - T_f) + 0.0039\, v\,(T_d - T_f)\right] \quad (9.15\ CU)$$

where,

- M = snowmelt runoff depth (in/day)
- C = coefficient (1 in most cases)
- v = wind speed 50 feet above snow (miles/hour)
- I_i = solar radiation (langleys/day)
- A = albedo (dimensionless)
- T_a = air temperature (^{o}F)
- T_f = temperature at which melt occurs (usually assumed to be 32^{o}F) (^{o}F)
- T_d = dew point temperature (^{o}F)

Shortwave radiation and longwave radiation are accounted for in the first term and second term, respectively. The third and fourth terms represent the effect of convection and condensation. Wind speed, solar radiation, and temperature data are used as inputs to the HEC-1 program. The program calculates albedo internally. Its value is based on the number of days since the last snowfall and varies from an initial value of 0.75 to a minimum value of 0.4. The program also automatically decreases the dew point temperature with elevation at a rate of 0.2 times the temperature lapse rate.

9.2.4.2 Degree-Day Method

The degree-day method further simplifies the relationship between snowmelt and the factors affecting snowmelt by developing a correlation analysis between temperature and snowmelt. The factors affecting snowmelt are either directly related to temperature or have some degree of correlation to temperature. The atmospheric temperature reflects the extent of radiation and the air's vapor pressure, and it is sensitive to wind. In addition, air temperature is frequently the only meteorological data available.

A degree-day indicates the amount of heat present to be create snowmelt and is defined as the deviation of the average daily temperature of 1 degree from a given datum temperature over a 24-hour period. The datum temperature for snowmelt calculations is normally 0^{o}C (32^{o}F). For example, if the average daily temperature is 5°C, the day would have 5 degree-days above freezing. The average daily temperature is sometimes taken as the average of the daily high and low temperatures.

The degree-day method correlates temperature and the amount of degree-days to snowmelt. A melt-rate coefficient is used to more accurately define the relationship between degree-days and snowmelt. The equation used in the degree-day method in the HEC-1 computer program is as follows (USACE, 1990):

$$M = C_m \left(T_a - T_f\right) \quad (9.16)$$

where,

M = snowmelt runoff depth, mm/day (in/day)
C_m = melt coefficient, mm/(day°C) (in/(day°F))
T_a = air temperature, °C (°F)
T_f = temperature at which melt occurs (usually assumed to be 0°C (32°F))

The degree-day is more valid for heavily forested areas where solar radiation and wind are less important in estimating snowmelt. The melt coefficient typically varies between 1.8 mm/°C to 3.7 mm/°C (0.04 in/°F to 0.08 in/°F). Higher values of the melt coefficient may be justified for time periods with high wind or high humidity.

9.2.4.3 Temperature Variation with Altitude

Air temperature generally decreases with elevation, holding all other factors constant. This temperature lapse rate is usually assumed to be 6.0°C per 1000 m (3.3°F per 1,000 ft). Since many of the snowmelt processes are temperature dependent it is important to divide watersheds with significant relief into elevation zones. These zones usually range from 200 to 400 m (650 to 1300 ft). Variable melting with altitude can be computed within the snowmelt models in HEC-1 by providing the temperature at the bottom of the lowest elevation zone, the temperatute lapse rate, and a specification of the altitude zones. The snowmelt is then estimated for each elevation zone, and an area-weighted average snowmelt is calculated for the entire watershed.

9.2.4.4 Runoff

Estimating runoff from snowmelt is important in estimating floods and operating flood control systems, operating river and storage facilities for transportation, environmental, and recreational purposes, and forecasting and managing water supplies for municipal, industrial, and agricultural purposes. Snowmelt is subject to the same losses as rainfall, mainly infiltration, and if rainfall is also occurring, the excess rainfall is combined with the excess snowmelt to estimate runoff. As has been previously discussed, unit hydrographs or other methods can then be used to generate runoff hydrographs.

9.3 ARID LANDS

Many parts of the Western United States are classified as arid or semiarid. The classification is based, in part, on the rainfall. However, vegetation and soils are also factors in classification. Generally speaking, arid lands are those where natural rainfall is inadequate to support crop growth. Semiarid lands are those where rainfall is only sufficient to support short-season crops.

From an engineering hydrology standpoint, arid and semiarid lands are characterized by little rainfall, which, when it does occur, is usually of an intense nature with runoff having a rapid response. Flash flooding is a major concern in such areas. Large amounts of sediment may also be produced as a part of flooding in these areas.

Hydrologic data are typically not available in arid and semiarid areas, at least in significant quantities. Where gages have been installed, the records are often characterized by years in which there is little or no rainfall and thus, no significant flooding. In other years, intense rainfalls of short duration produce high peak discharges relative to the total volume of runoff. These factors make it comparatively difficult to provide estimates of flood magnitudes or probabilities.

Another feature of many arid locations is the alluvial fan. Alluvial fans are briefly described in this section along with an overview of the stages of assessment for alluvial fan flooding and their relevance for highway design.

9.3.1 Gaged Flow Analysis

Annual floods in arid regions often closely follow a log-normal or extreme value distribution. Log Pearson Type III curve fitting techniques are also applicable, provided that the annual series of peaks has non-zero values. However, annual maximum flood records that include values of zero are not uncommon in arid regions. Thus, a frequency curve based on logarithms, such as the log Pearson Type III, must be adapted to since the logarithm of zero is minus infinity. In such cases, USGS Bulletin 17B (1982) provides a method for computing a frequency curve for records that includes zero-flood years. The method is based on the method of Jennings and Benson (1969). The method is referred to as the conditional probability adjustment. When this method is applied, three frequency curves are computed: the initial or unadjusted curve, the conditional frequency curve, and the synthetic frequency curve. The selection from among the curves to make estimates of flood magnitudes depends on the assessment of the hydrologist.

The procedure to follow when analyzing records that include zero-flood years consists of the following six steps:

1. Preliminary analysis
2. Check for outliers
3. Compute unadjusted frequency curve
4. Compute conditional frequency curve
5. Compute synthetic frequency curve
6. Select frequency curve to make estimates

Each of these steps is discussed in detail.

Step 1. Preliminary Analysis. The first step in the analysis is to separate the record into two parts, all non-zero floods and zero floods. The procedures can only be applied when the number of zero-flood years does not exceed 25 percent of the total record length; thus,

$$\frac{n_z}{N_t} \leq 0.25 \qquad (9.17)$$

where,

n_z =number of zero-flood years

N_t = total record length including those years with zero-floods

After eliminating the zero-flood years, the mean, standard deviation, and standardized skew coefficient of the logarithms are computed with the remainder of the data. The skew should be rounded to the nearest tenth.

Step 2. Check for Outliers. The test for outliers from USGS Bulletin 17B (1982) is discussed in Section 4.3.6.1. While low outliers are more common than high outliers in flood records from arid regions, tests should be made for both. The procedure depends on the station skew. If low outliers are identified, then they are censored (i.e., deleted from the flood record) and the moments recomputed. When high outliers are identified, the moments must be recomputed using the historic-peak adjustment.

Step 3. Compute Unadjusted Frequency Curve. The moments of the logarithms from step 1, or from step 2 if outliers were identified, are used to compute the unadjusted frequency curve. For this step, station skew rather than weighted skew should be used. For selected exceedence probabilities, values of the log-Pearson Type III deviates (K) are obtained from Table 4.13 for the station skew. The deviates are then used with the log mean ($\bar{Y}$) and log standard deviation (S_y) to compute the logarithm of the discharge:

$$Y = \bar{Y} + K\,S_y \tag{9.18}$$

The frequency curve can be plotted on log-probability scales using the Y values and the exceedence probabilities, P_e, used to obtain the corresponding values of K. The data points can be plotted using a plotting position formula such as the Cunnane or Weibull.

Step 4. Compute Conditional Frequency Curve. To derive the conditional frequency curve, the conditional exceedence probability (P'_e) is computed as:

$$P'_e = (p_c)\,(P_e) \tag{9.19}$$

where,

P_e = unadjusted exceedence probability

p_c = zero flow adjustment

p_c is estimated as follows:

$$p_c = \frac{N_t - n_z}{N_t} = \frac{\text{number of floods excluding zero values}}{\text{total number of years in record}} \tag{9.20}$$

If historic information is available, then the conditional probability should be computed by Equation 9.21 rather than Equation 9.20:

$$p_c = \frac{H - W\,L}{H} \tag{9.21}$$

where,

H	=	historic record length
L	=	number of peaks truncated
W	=	systematic record weight

The probability p_c is then multiplied by each probability used in Step 3 to obtain the K values and plot the unadjusted frequency curve in Step 3. The adjusted probabilities and the discharges computed with Equation 9.18 are plotted on frequency scales to form the conditional frequency curve. If the curve is plotted on the same scales as the unadjusted curve from Step 3, then the conditional frequency curve can be compared to the measured data points.

Step 5. Compute Synthetic Frequency Curve. The conditional frequency curve of Step 4 does not have known moments. Approximate values, which are referred to as synthetic statistics, can be computed. Since there are three moments, three points on the conditional frequency curve will be used to fit the synthetic frequency curve. Specifically, the discharge values for exceedence probabilities of 0.01, 0.1, and 0.50 are used. Discharges, obtained from the conditional frequency curve, are used to compute the synthetic statistics generated by the following equations:

$$G_s = -2.5 + 3.12\ \log(Q_{0.01} / Q_{0.10}) / \log(Q_{0.10} / Q_{0.50}) \tag{9.22}$$

$$S_s = \log(Q_{0.01} / Q_{0.50}) / (K_{0.01} - K_{0.50}) \tag{9.23}$$

$$\overline{Y_s} = \log(Q_{0.50}) - K_{0.50}(S_s) \tag{9.24}$$

in which $K_{0.50}$ and $K_{0.01}$ are the log-Pearson Type III deviates obtained from Table 4.13 for the synthetic skew G_s. Equation 9.22 for the synthetic skew is an approximation for use between skew values of -2.0 and +2.5. If appropriate, the synthetic skew can be used to compute a weighted skew, which would be used in place of the synthetic skew.

The synthetic statistics can then be used in the following formula to compute the synthetic frequency curve.

$$Q = 10^{\overline{Y}_s + KS_s} \tag{9.25}$$

When verifying the synthetic curve, the plotting positions for the synthetic curve should be based on either the total number of years of record or the historic record length H, if the historic adjustment is used.

Step 6. Select a Curve to Make Estimates. The first five steps have resulted in three frequency curves: the unadjusted, the conditional, and the synthetic curves. All three are of potential value for making flood estimates. Each should be compared to the measured data and the goodness of fit assessed.

The disadvantages of the unadjusted curve are that it uses station skew, which can be highly variable for small record lengths, and that it does not account for the zero years in the record, which can be significant if the number of zero-flood years is relatively large. The adjustment with Equation 9.20 or Equation 9.21 is an attempt to overcome the lack of accountability for zero-flood years, but since the zero-flood years are essentially years of low rainfall, applying the adjustment of Equation 9.20 to the high-flow years may produce a distortion on the high end of the curve.

The disadvantages of the synthetic curve are that it depends on three exceedence probabilities, which have been selected conceptually, and that the form of Equations 9.22, 9.23, and 9.24 are subjective. These disadvantages should be considered when selecting one of the curves to make estimates.

<u>Example</u>. Table 9.8 contains the annual maximum discharge record (1932-1973) for Orestimba Creek near Newman, CA (USGS Gauging Station 11-2745). This record was analyzed in USGS Bulletin 17B (1982) and includes 6 years in which there was no discharge. To ensure that the adjustment method is applicable, the ratio of the number of zero-flood years (n_z) to the total record length (N_t) must be less than or equal to 0.25. In this case, the method can be applied since 6/42 equals 0.143.

The six zero values are dropped from the record, which gives $n = N_t - n_z = 36$, and the moments of the logarithms are then computed:

Variable	**Value in SI**	**Value in CU**
Mean of the logarithms	1.53	3.08
Standard deviation of the logarithms	0.64	0.64
Station skew coefficient of the logarithms	-0.84	-0.84

The skew is rounded to the nearest tenth, which in this case is -0.8.

The remaining record (n = 36) should be checked for outliers. The procedure detailed in Chapter 4 is used. Since the skew is less than -0.4, the record is first checked for low outliers. For a 36-year record length, an outlier deviate (K_n) of 2.639 is obtained from Table 4.21. The logarithm of the critical flow for low outliers is computed as follows:

	Value in SI	Value in CU
$\log Q_0 = \bar{X} - K_nS$	= 1.53 - 2.639(0.64)= -0.16	= 3.08 - 2.639(0.64) = 1.39
Critical flow, Q_O	$=10^{-0.16} = 0.7\ m^3/s$	$=10^{1.39} = 25\ ft^3/s$

Because the 1955 flow of 0.45 m^3/s (16 ft^3/s) is less than this critical flow, it is considered a low outlier. The value is censored and the remaining 35 values are used to compute the following moments of the logarithms:

	Value in SI	Value in CU
Mean of the logarithms	1.58	3.13
Standard deviation of the logarithms	0.57	0.57
Station skew of the logarithms	-0.44	-0.44

According to the flow chart for handling outliers, it is next necessary to check for high outliers. The procedure described in Chapter 4 is used. For a sample size of 35, the outlier deviate (K_n) from Table 4.21 is 2.628. Thus, the logarithm of the critical flow for high outliers is:

	Value in SI	Value in CU
$\log Q_0 = \bar{X} + K_nS$	= 1.58 + 2.628(0.57)= 3.08	= 3.13 + 2.628(0.57) = 4.63
Critical flow, Q_O	$=10^{3.08} = 1200\ m^3/s$	$=10^{4.63} = 43{,}000\ ft^3/s$

None of the flows in the record exceeded the critical flow; thus, there are no high outliers.

Table 9.8. Annual Maximum Flood Series: Orestimba Creek, CA (Station 11-2745)

	SI		CU		Exceedence Plotting Probability
Year	Flow (m^3/s)	Log of flow	Flow (ft^3/s)	Log of Flow	
1932	120.6	2.081	4,260	3.629	0.222
1933	9.8	0.991	345	2.538	0.806
1934	14.6	1.164	516	2.713	0.750
1935	37.4	1.573	1,320	3.121	0.556
1936	34.0	1.531	1,200	3.079	0.611
1937	61.7	1.790	2,180	3.338	0.417
1938	91.5	1.961	3,230	3.509	0.333
1939	3.3	0.519	115	2.061	0.972
1940	97.4	1.989	3,440	3.537	0.306
1941	86.9	1.939	3,070	3.487	0.361
1942	53.2	1.726	1,880	3.274	0.444
1943	182.7	2.262	6,450	3.810	0.083
1944	36.5	1.562	1,290	3.111	0.583
1945	169.1	2.228	5,970	3.776	0.111
1946	22.1	1.344	782	2.893	0.667
1947	0.0	*	0	*	
1948	0.0	*	0	*	
1949	9.5	0.978	335	2.525	0.833
1950	5.0	0.699	175	2.243	0.861
1951	82.7	1.918	2,920	3.465	0.389
1952	103.7	2.016	3,660	3.563	0.278
1953	4.2	0.623	147	2.167	0.917
1954	0.0	*	0	*	
1955	0.45	+	16	+	
1956	159.2	2.202	5,620	3.750	0.139
1957	40.8	1.611	1,440	3.158	0.528
1958	288.9	2.461	10,200	4.009	0.028
1959	152.4	2.183	5,380	3.731	0.167
1960	12.7	1.104	448	2.651	0.778
1961	0.0	*	0	*	
1962	49.3	1.693	1,740	3.241	0.472
1963	235.1	2.371	8,300	3.919	0.056
1964	4.4	0.643	156	2.193	0.889
1965	15.9	1.201	560	2.748	0.722
1966	3.6	0.556	128	2.107	0.944
1967	118.9	2.075	4,200	3.623	0.250
1968	0.0	*	0	*	
1969	143.9	2.158	5,080	3.706	0.194
1970	28.6	1.456	1,010	3.006	0.639
1971	16.5	1.217	584	2.766	0.694
1972	0.0	*	0	*	
1973	42.8	1.631	1,510	3.179	0.500

* Zero-flow year + Low outlier

The unadjusted curve is computed using the 35 values. The mean, standard deviation, and skew of the logarithms are shown above. The skew is rounded to -0.4. The computations of the unadjusted curve are given in Table 9.9, and the curve is shown in Figure 9.17, with the values of column 4 of Table 9.9 plotted versus the exceedence probabilities of column 1.

Table 9.9. Computation of Unadjusted and Conditional Frequency Curves

		SI		CU		
(1) Exceedence Probability P_e	(2) Log-Pearson Type III Deviate (K) for g = -0.4	(3) log Q	(4) $Q(m^3/s)$	(3) log Q	(4) $Q(ft^3/s)$	(5) Adjusted Exceedence Probability
0.99	-2.61539	0.089	1.2	1.639	44	0.825
0.9	-1.31671	0.829	6.8	2.379	240	0.750
0.7	-0.47228	1.311	20.5	2.861	726	0.583
0.5	0.06651	1.618	41.5	3.168	1,472	0.417
0.2	0.85508	2.067	116.8	3.617	4,144	0.167
0.1	1.23114	2.282	191.3	3.832	6,788	0.083
0.04	1.60574	2.495	312.8	4.045	11,100	0.033
0.02	1.83361	2.625	421.8	4.175	14,970	0.017
0.01	2.02933	2.737	545.4	4.287	19,350	0.0083
0.002	2.39942	2.948	886.5	4.498	31,450	0.0017

(SI) (column 3) $\log Q = \overline{X} + KS = 1.58 + 0.57\,K$
(CU) (column 3) $\log Q = \overline{X} + KS = 3.13 + 0.57\,K$

Using the statistics for the censored series with n = 35, the conditional frequency curve is computed using the conditional probability adjustment. Log-Pearson III deviates are obtained from Table 4.13 for a skew of -0.4 and selected exceedence probabilities (see Table 9.9). Since there are 35 events remaining after removing the zero flows and the low outlier, the expected probability of Equation 9.21 is 35/42 = 0.8333.

The frequency curves with and without the conditional probability adjustment are shown in Figure 9.17. The conditional curve graphs the flow of column 4 of Table 9.9 versus the probability from column 5. The measured data (n = 35) are also plotted in Figure 9.17. Neither curve provides a good representation of the data in the lower tail.

The synthetic statistics can be computed using Equations 9.22, 9.23, and 9.24. These require values of discharges from the adjusted frequency curve for exceedence probabilities of 0.01, 0.1, and 0.5, which are denoted as $Q_{0.01}$, $Q_{0.10}$, and $Q_{0.50}$, respectively. These three values must be estimated graphically because the probabilities do not specifically appear in the computations (column 5) of Table 9.9. There is no known mathematic equation that represents the adjusted curve. The values from the adjusted curve of Figure 9.17 are as follows: $Q_{0.01}$ = 510 m^3/s (18,000 ft^3/s), $Q_{0.10}$ = 170 m^3/s (6,000 ft^3/s), and $Q_{0.50}$ = 30 m^3/s (1,060 ft^3/s), respectively. Thus, the synthetic skew, standard deviation, and mean are as follows:

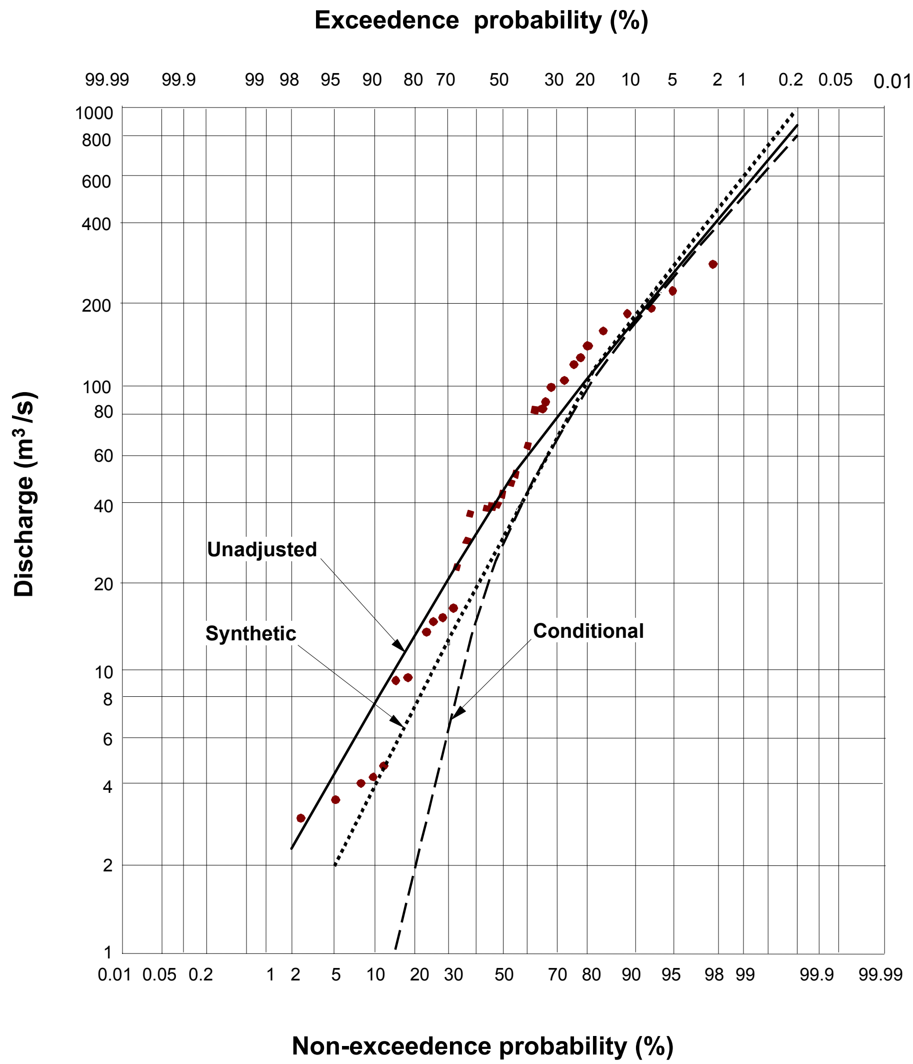


Figure 9.17. Unadjusted, conditional, and synthetic frequency curves, Orestimba Creek, CA

	Value in SI	Value in CU
G_s	$= -2.50 + 3.12\left[\dfrac{\log(510/170)}{\log(170/30)}\right] = -0.52$	$= -2.50 + 3.12\left[\dfrac{\log(18{,}000/6000)}{\log(6000/1060)}\right] = -0.52$
S_s	$= \dfrac{\log(510/30)}{1.95472 - 0.08302} = 0.66$	$= \dfrac{\log(18{,}000/1060)}{1.95472 - 0.08302} = 0.66$
$\overline{X}_S$	$= \log(30) - 0.08302(0.66) = 1.42$	$= \log(1060) - 0.08302(0.66) = 2.97$

The computed skew value of -0.52 should be rounded to the nearest tenth; thus, G_s = -0.5. The values of $K_{0.01}$ and $K_{0.50}$ for calculation of the standard deviation are obtained from Table 4.13 using the synthetic skew of -0.5.

The weighted skew is used with the synthetic mean and synthetic standard deviation to compute the final frequency curve. The generalized skew coefficient for the location of the gage is -0.3, with a mean square error of 0.302. The mean square error for the synthetic skew, which is obtained from Table 4.7, is 0.163. Thus, the weighted skew is:

$$G_w = \frac{0.302(-0.52) + 0.163(-0.3)}{0.302 + 0.163} = -0.44$$

This can be rounded to the nearest tenth; thus, G_w = -0.4, which is used to obtain the deviate K values from Table 4.13. The computations are provided in Table 9.10. The synthetic curve is also plotted in Figure 9.17.

Table 9.10. Computation of the Synthetic Frequency Curve

		SI		CU	
(1) Exceedence Probability P_e	**(2) log-Pearson Type III Deviate (K) for g = -0.4**	**(3) log Q**	**(4) Q (m^3/s)**	**(3) log Q**	**(4) Q (ft^3/s)**
0.99	-2.61539	-0.306	0.5	1.244	18
0.9	-1.31671	0.551	3.6	2.101	126
0.7	-0.47228	1.108	13	2.658	455
0.5	0.06651	1.464	29	3.014	1,030
0.2	0.85508	1.984	96	3.534	3,420
0.1	1.23114	2.233	171	3.783	6,060
0.04	1.60574	2.480	302	4.030	10,700
0.02	1.83361	2.630	427	4.180	15,100
0.01	2.02933	2.759	575	4.309	20,400
0.002	2.39942	3.004	1,008	4.554	35,800

(SI) (column 3) $\log Q = \overline{X}_s + KS_s = 1.42 + 0.66\,K$
(CU) (column 3) $\log Q = \overline{X}_s + KS_s = 2.97 + 0.66\,K$

None of the three curves closely follow the trend in the measured data, especially in the lower tail. However, reasonably good agreement is found in the upper portion where design values are generally required. The synthetic curve is based, in part, on the generalized skew, which is the result of regionalization of values from watersheds that may have different hydrologic characteristics than those of Orestimba Creek. In order to make estimates of flood magnitudes, one of the curves must be selected. This would require knowledge of the watershed and judgment of the hydrologist responsible for the analysis.

9.3.2 Regression Equations for Southwestern U.S.

The USGS (Thomas, et al., 1993) provides regression equations for the southwestern U.S. These equations are also part of the National Flood Frequency Program (see Chapter 5).

9.3.2.1 Purpose and Scope

The report describes the results of a study to develop reliable methods for estimating the magnitude and frequency of floods for gaged and ungaged streams in the southwestern United States and to improve the understanding of flood hydrology in the southwestern United States.

The large study area, which encompasses most of the arid lands of the southwestern United States, includes all of Arizona, Nevada, and Utah, and parts of California, Colorado, Idaho, New Mexico, Oregon, Texas, and Wyoming. The study area was further divided into 16 flood regions (see Figure 9.18).

The data examined in the study includes sites with drainage areas of less than 5,200 km^2 (2,000 mi^2) and mean annual precipitation of less than 1,730 mm (68.1 in). The focus of the study, however, was on drainage areas of less than about 500 km^2 (200 mi^2) and arid areas with less than 510 mm (20 in) of mean annual precipitation. The series of annual peak discharges for sites used in this study are unaffected by flow regulation, and the individual sites have at least 10 years of record through water year 1986. The lower end of applicability of the equations varies by region.

The basic regional method used in this study is an information-transfer method in which flood-frequency relations determined at gaged sites are transferred to ungaged sites using multiple-regression techniques. Flood-frequency relations were determined at gaged sites using guidelines recommended in USGS Bulletin 17B (1982). Ordinary and generalized least-squares multiple-regression analyses were used to relate the gaged-site flood-frequency relations to basin and climatic characteristics.

The regional study offers several advantages compared with previous state-wide regional studies. The large database of more than 1,300 gaged sites with about 40,000 station years of annual maximum peaks can decrease the time-sampling error of flood estimates, which can be a problem with small data sets in the southwestern United States. Some of the recent regional studies developed for single states have large differences in the estimated flood-frequency relations at state boundaries. These different estimates of flood magnitudes at state boundaries were removed in this study. Regional relations that were derived from the large database with a large range of values are potentially more reliable than relations derived from smaller databases and can be used with less extrapolation for ungaged streams.

9.3.2.2 Description of Study Area

The study area is over 1.5 million km^2 (580,000 mi^2). The area is bounded by the Rocky Mountains on the east, the northern slopes of the Snake River basin on the north, the Cascade-Sierra Mountains on the west, and the international border with Mexico on the south. The Basin and Range province in the western and southern part of the study area has mostly isolated block mountains separated by aggraded desert plains. The mountains commonly rise abruptly from the valley floors and have piedmont plains that extend downward to neighboring basin floors. Several large flat desert areas are interspersed between the mountains, and some are old lake bottoms that have not been covered with water for hundreds of years. Many of the piedmont plains contain distributary-flow areas that are composed of material deposited by mountain-front runoff.

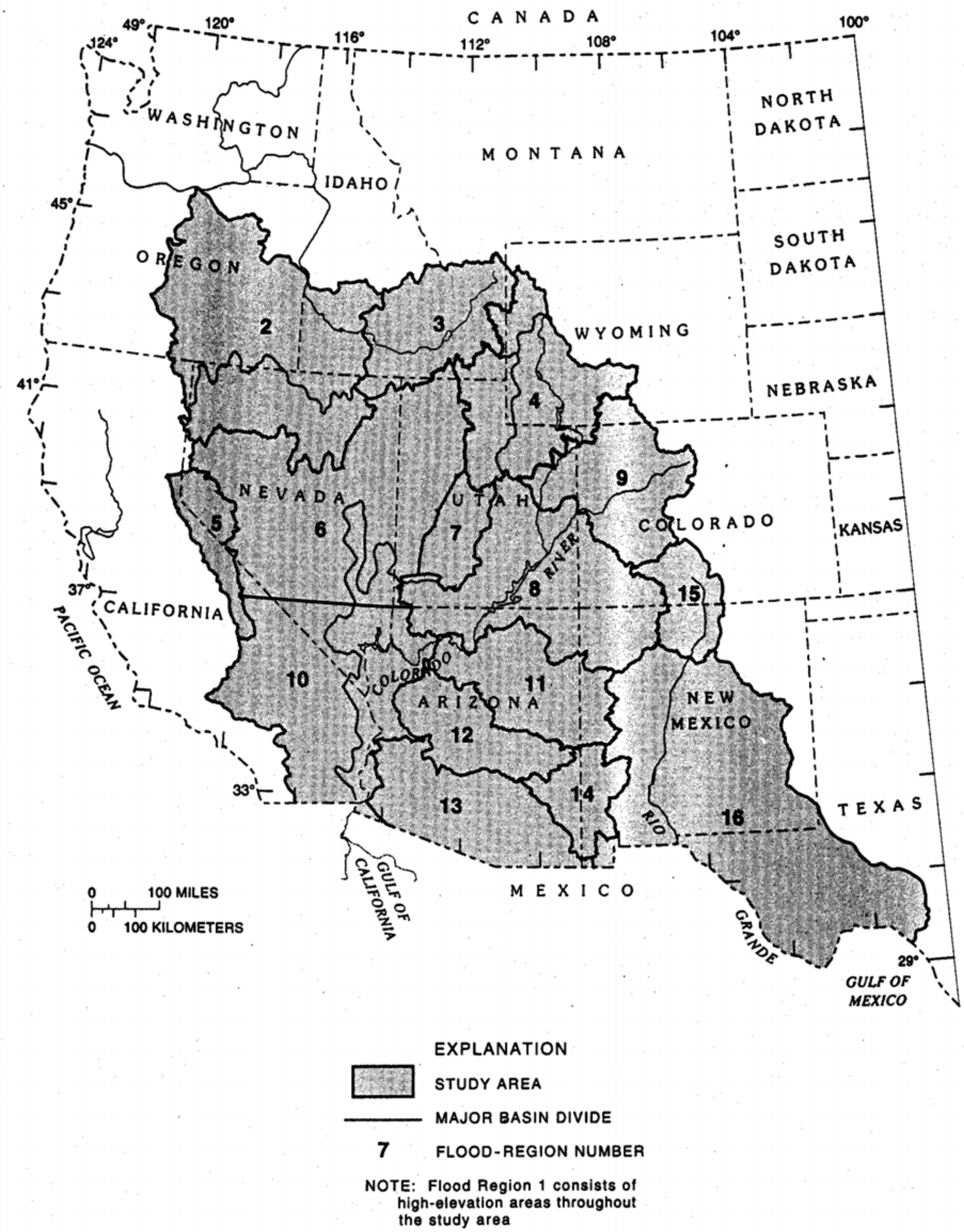


Figure 9.18. Flood Regions in Study Area (Thomas, et al., 1997)

Most of the streams in the study area flow only in direct response to rainfall or snowmelt. In the northern latitudes and at the higher elevations where the climate is cooler and more humid, most of the streams flow continuously. Streams in alluvial valleys and base-level plains are perennial or intermittent where the stream receives ground-water outflow. Small streams in the southern latitudes commonly flow only a few hours during a year.

An arid or semiarid climate in the middle latitudes exists where potential evaporation from the soil surface and from vegetation exceeds the average annual precipitation. About 90 percent of the study area is arid or semiarid and has a mean annual precipitation of less than 510 mm (20 in). In addition to the generally meager precipitation, the climate of the study area is characterized by extreme variations in precipitation and temperature. Mean annual precipitation ranges from more than 1,270 mm (50 in) in the Cascade-Sierra Mountains in California to less than 80 mm (3.1 in) in the deserts of southwestern Arizona and southeastern California. Temperatures range from about 43^{o}C (109^{o}F) in the southwestern deserts in the summer to below -18^{o}C (0^{o}F) in the northern latitudes and mountains in the winter. Precipitation in the study area is variable temporally and spatially. In some extremely arid parts of the study area, the mean annual precipitation has been exceeded by the rainfall from one or two summer thunderstorms.

9.3.2.3 Peak Discharge Equations

Equations for estimating 2-, 5-, 10-, 15-, 50-, and 100-year peak discharges at ungaged sites in the southwestern United States were developed using generalized least-squares, multiple-regression techniques and a hybrid method that was developed in this study. The equations are applicable to unregulated streams that drain basins of less than about 500 km^2 (200 mi^2). Drainage area, mean basin elevation, mean annual precipitation, mean annual evaporation, latitude, and longitude are the basin and climatic characteristics used in the equations. The equations for one region are given in Table 9.11 as an illustration.

Detailed flood-frequency analyses were made of more than 1,300 gauging stations with a combined 40,000 station years of annual peak discharges through water year 1986. The log-Pearson Type III distribution and the method of moments were used to define flood-frequency relations. A low-discharge threshold was applied to about one-half of the sites to adjust the relations for low outliers. With few exceptions, the use of the low-discharge threshold resulted in markedly better appearing fits between the computed relations and the plotted annual peak discharges. After all adjustments were made, 80 percent of the gauging stations were judged to have adequate fits of computed relations to the plotted data. The individual flood-frequency relations were judged to be unreliable for the remaining 20 percent of the stations because of extremely poor fits of the computed relations to the data, and these relations were not used in the generalized least-squares regional regression analysis. Most of the stations with unreliable relations were from extremely arid areas with 43 percent of the stations having no flow for more than 25 percent of the years of record. A new regional flood-frequency method, which is named the hybrid method, was developed for those more arid regions.

An analysis of regional skew coefficient was made for the study area. The methods of attempting to define the variation in skew by geographic areas or by regression with basin and climatic characteristics all failed to improve on a mean of zero for the sample. The regional skew used in the study, therefore, was the mean of zero with an associated error equal to the sample variance of 0.31 log units.

The general form for the equations for High-Elevation Region 1 is:

$$Q_T = a_T A^{b_{1T}} P^{b_{2T}} \tag{9.26}$$

where,

Q_T = peak discharge for return period T, m^3/s (ft^3/s)
A = drainage area, km^2 (mi^2)
P = mean annual precipitation, mm (in)
a_T, b_{1T}, and b_{2T} = constants summarized in Table 9.11.

Table 9.11. Generalized Least-Squares Regression Equations for Estimating Regional Flood-Frequency Relations for the High-Elevation Region 1

Return Period, T (years)	a_T (SI)	a_T (CU)	b_{1T}	b_{2T}	Avg. Standard Error (%)	Equivalent Record Length (years)
2	1.49E-05	0.124	0.845	1.44	59	0.16
5	2.21E-04	0.629	0.807	1.12	52	0.62
10	8.64E-04	1.43	0.786	0.958	48	1.34
25	3.05E-03	3.08	0.768	0.811	46	2.50
50	6.13E-03	4.75	0.758	0.732	46	3.37
100	1.08E-02	6.78	0.750	0.668	46	4.19

(from Thomas, et al., 1997)

9.3.3 Transmission Losses

When the initial part of a runoff hydrograph enters and flows through a dry stream channel, significant amounts of water can seep into the bed and banks of the stream. This seepage is called transmission loss. Transmission loss rates vary widely over the duration of a flood hydrograph and throughout a region. Such losses are important because they can significantly change the shape of a hydrograph and because the volume of seepage can reduce the volume of flow at downstream channel sections.

The amount of losses depend on the material characteristics of the stream cross-section, the surface area of the beds and banks of the reach, the location of the ground-water table, antecedent moisture of the cross-section, and the existence and type of vegetation in the stream. The latter two factors are usually not considered in design work, but may need to be considered in the analysis of data.

For conditions in which there is observed inflow and outflow data, no uniform lateral inflow, and no out-of-bank flow, the following methodology may be used to estimate transmission losses. This methodology is discussed in detail in Chapter 19 of the National Engineering Handbook, and it should be consulted for details of the assumptions and limitation of this methodology (Lane, 1983).

This method estimates the outflow volume Q_d at the end of a reach given the volume at the upper end of the reach, Q_u. Where measured data from previous storm events are available, a linear water yield model is used:

$$Q_d = \begin{cases} 0 & for\ Q_u \le Q_o \\ a + b\,Q_u & for\ Q_o < Q_u > Q_1 \\ Q_u - V & for\ Q_1 \le Q_u \end{cases} \tag{9.27}$$

where,

a, b = regression coefficients
V = maximum potential loss
Q_1 = maximum loss threshold volume
Q_o = minimum loss threshold volume, computed as:

$$Q_o = \frac{-a}{b} \tag{9.28}$$

This method requires the following constraints on the regression coefficients:

$$\begin{aligned} a &\le 0 \\ 0 < b &\le 1 \end{aligned} \tag{9.29}$$

If these constraints are not met, the data should be examined to detect data points that may cause the irrationality. Graphical analysis is useful for identifying data points that may be questionable.

The corresponding peak discharge is computed by:

$$q_d = \begin{cases} 0 & if\ Q_d = 0 \\ \dfrac{(Q_d - Q_u)}{D} + b'\,q_u & if\ Q_d > 0 \end{cases} \tag{9.30}$$

where,

b' = adjusted regression slope (b' = b if Q_u<Q_1)
D = duration of the inflow
q_u = peak rate of inflow at the upper reach

The linear regression parameters can be estimated from the measured data as

$$a = \overline{Q}_d - b\overline{Q}_u \tag{9.31}$$

$$b = \frac{\sum_{i=1}^{n} [(Q_{di} - \overline{Q}_d)(Q_{ui} - \overline{Q}_u)]}{\sum_{i=1}^{n} (Q_{ui} - \overline{Q}_u)^2} \tag{9.32}$$

where,

$\overline{Q}_d$ = mean outflow volume

$\overline{Q}_u$ = mean inflow volume

Lane (1983) provides extensions of this method such that lateral inflow can be accounted for and for sites where gaged data are not available.

9.3.4 Alluvial Fans

Alluvial fans in arid and semi-arid environments can be defined as a "sedimentary deposit located at a topographic break, such as the base of a mountain front, escarpment, or valley side, that is composed of fluvial and/or debris flow sediments and which has the shape of a fan either fully or partially extended" (National Research Council, 1997). Knowledge and expertise in hydrology, open channel hydraulics, geology, and geomorphology are important understand the processes occurring in the formation of alluvial fans.

The creation of an alluvial fan requires a source of sediment and debris and the means to convey this material to the depositional area. In the depositional area the sediment carrying capacity of the stream is reduced due to an increased flow area.

Fan features include the topographic apex, which is the head of highest point on an active alluvial fan, and the hydrographic apex, which is the highest point on an alluvial fan where flow is last confined. On an active alluvial fan, the flow paths are uncertain and may diverge and/or rejoin as is shown in Figure 9.19.

Flow paths may shift during each flow event and between flow events. Flows may be debris flows, water flows, or a mixture. These shifts and complex flows add significant design challenges for the highway designer crossing an alluvial fan. The following discussion provides an introduction to the subject of alluvial fans. The reader should consult the more comprehensive resources cited in the references for a more detailed treatment.

9.3.4.1 Assessment of Alluvial Fans

There are three phases to assessing the hydrologic and hydraulic characteristics of alluvial fans. The first phase is to identify the presence of alluvial fans within the project area. They can be identified from soils maps, geologic maps, topographic maps, and aerial photographs. In addition to these sources, a site visit is invaluable in defining alluvial fans and their characteristics.

The second phase is defining the active and inactive regions of the alluvial fan. Inactive regions on the alluvial fans may be covered with vegetation and the channel will be incised and capable of carrying the design flow under the given conditions. Active areas will have newer sediment deposits and a relative lack of vegetation. Larger flows or different conditions may allow the flow to break out of the channel in a process called avulsion. The flow and new channel may cross the project area at a new location.

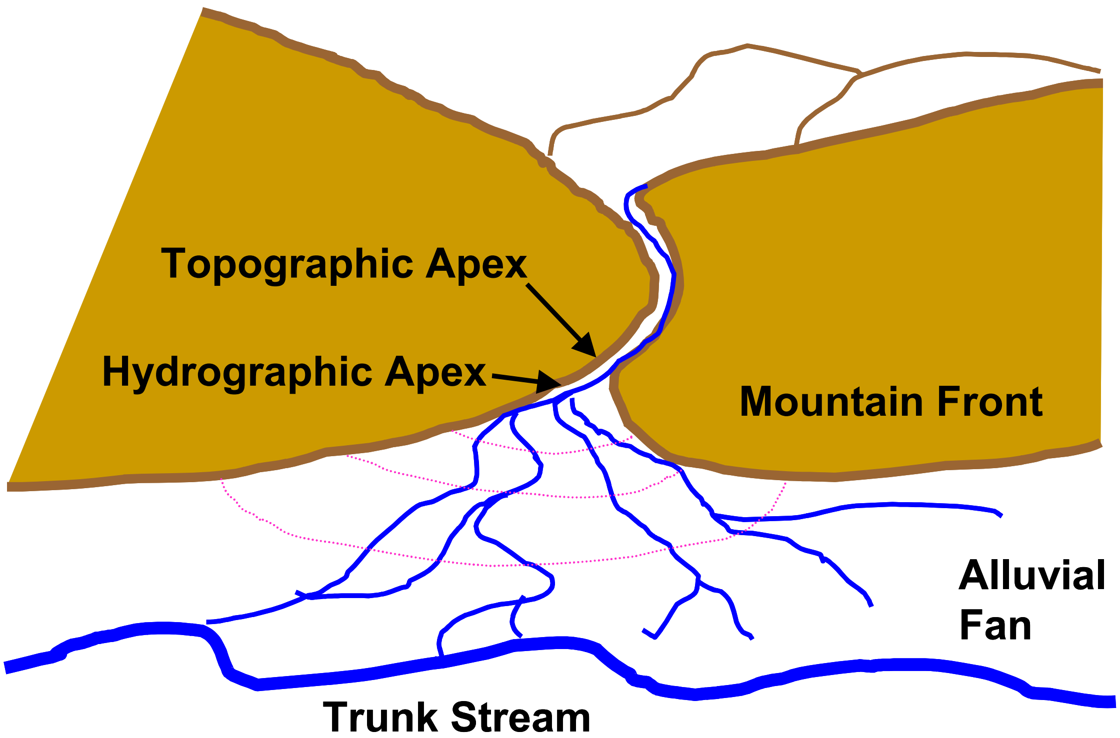


Figure 9.19. Schematic of an Alluvial Fan

The third phase of assessing alluvial fans is to define the design flow at a given point on the fan. This is a function of not only the precipitation and factors affecting runoff, but also the probability of flooding at any location on the active portion of the fan. The sediment content of a flow may vary from negligible sediment to more than 50% sediment and debris, bulking the flow, and creating the need to design channel crossings and other structures for this increased flow volume. An assessment of the conditional probability of flooding at all locations across the active portion of the fan will assist in determining the flood at a particular location, the T-year flood.

9.3.4.2 Flood Estimation

The determination of the magnitude and location of flooding on alluvial fans is beyond the scope of this document. For a more detailed discussion of processes affecting flow on alluvial fans, methodologies that can be employed to estimate the magnitude and location of alluvial fan flooding, and computer models that can be utilized in the prediction of alluvial fan flooding, the following references can be used.

FEMA's *Guidelines for Determining Flood Hazards on Alluvial Fans* discusses the three phases in alluvial fan flood assessment. The U.S. Army Corps of Engineers (USACE) has developed a methodology and computer program that uses the principles of risk-based analyses to estimate flood hazards on alluvial fans. The methodology is discussed in *Guidelines of Risk and Uncertainty Analysis in Water Resources Planning* (USACE, 1992). FEMA has developed a computer program called FAN that performs this methodology. It is provided and discussed in *FAN, An Alluvial Fan Flooding Computer Program User's Manual and Program Disk* (FEMA, 1990). Two-dimensional models may also be used to model flow on alluvial fans. These models can estimate the characteristics of flows with a large amount of sediment, unconfined flow, split flow, mud and debris flow, and complex urban flooding. The *Flood Insurance Study Guideline and Specifications for Study Contractors* provides guidelines and several methods applicable to conditions found on alluvial fans (FEMA, 1995). The USACE's *Assessment of Structural Flood-Control Measures on Alluvial Fans* lists several types of flood control measures used on alluvial fans and their advantages and disadvantages, and it provides several case studies of their application.

9.4 ADVANCED APPLICATIONS

A simple model, such as the rational method, will continue to be sufficient to meet the design requirements of many drainage structures. In a growing number of cases, however, the hydrologic aspects of drainage design are too complex to be met with a model as simple as the rational method. In addition to ensuring the safety of the structure during flood conditions, the transportation hydrologist must often consider issues such as stormwater management, floodplain inundation, and broad environmental impacts. Analysis of these issues generally requires the use of a complete runoff hydrograph and detailed mapping. In an ungaged watershed, complete hydrographs can be synthesized with physically-based models that simulate and link the individual runoff processes taking place in the watershed.

Many physically-based models can be used to develop a runoff hydrograph. Most have a module that estimates the depth of runoff, or rainfall excess, as a function of the rainfall and the spatial distribution of the land cover and soils in the watershed. The SCS curve number procedure described in Chapter 5 is an example of such an approach. The runoff hydrograph is then developed by routing the time and spatially varying rainfall excess across the land surfaces and through the stream network.

Physically-based models that have parameters defined in terms of the spatial distribution of the watershed land cover and drainage network are very attractive. The hydrologist can vary the model parameters to simulate the behavior of the watershed under existing or future development conditions or to examine the consequences of an array of design options. A major problem with the use of any physically-based model is that the definition of the model parameters is usually a difficult, tedious, time-consuming, and expensive task. While the execution of the model's computer program may take seconds, it may take weeks of map manipulations and table look-ups to define the input parameters when the watershed is large.

If the hydrologist is to use the complex models that are being required with increasing frequency, the input data must be developed with the same efficiency and quality control as can be accomplished in the computer execution of the model. The tools available in the field of geographic information systems (GIS) can be used to define map-based input parameters in a fraction of the time required by traditional approaches. For example, a GIS can be used to develop and store a digital database containing the land cover, soil type and topography for a state or county highway department's entire area of jurisdiction. After the data are acquired, a properly configured GIS often allows the user to meet the office phases of the modeling tasks without leaving the desk.

9.4.1 Watershed Modeling

Although many streams have been gaged to provide a record of streamflow over time, most streams encountered in highway drainage do not have available streamflow information. Precipitation data, however, are relatively abundant and numerous models are available that allow the determination of runoff. Planners and engineers must rely on synthesis and simulation as tools to generate synthetic flow sequences used for design discharge rates and decision-making regarding the effects of land use, urban planning, flood control measures, water supply, and water quality. Simulation is defined as the mathematical description of a real system and imitate the behavior of the system. A hydrologic simulation model is a set of equations and algorithms that describe the response of a hydrologic water resource system to a series of events during a selected time period and is commonly used in generating streamflow hydrographs from rainfall data and watershed characteristic data.

Watershed modeling centers on readily available computer programs such as the U.S. Department of Agriculture's TR-20 (SCS, 1984), the EPA Storm Water Management Model (Huber and Dickinson, 1988; Roesner, Aldrich and Dickinson, 1988) and the US Army Corps of Engineers' HEC-1 (HEC, 1985) and HEC-HMS (HEC,2000). Another approach is to use the computer programs in the Federal Highway Administration's Integrated Drainage Design Computer System - HYDRAIN (Young and Krolak, 1992) or the Watershed Modeling System (Nelson, 2001). After the input parameters have been entered into any of these models, procedures that would require days to be executed through hand calculations are completed in seconds or minutes.

9.4.1.1 The Modeling Process

The modeling process can be divided into three phases: identification, conceptualization, and implementation. The identification phase analyzes existing and proposed components of the system to be studied and collects all pertinent data to by used in the analysis. Information that may be necessary are subwatershed characteristics, channel characteristics, meteorological data, water use information, streamflow data, and reservoir/storage information.

The conceptualization phase identifies system components that are important to define the behavior of the system and it frequently provides feedback to the identification phase by defining actual data requirements. This phase chooses the techniques to be used to represent the system elements, and selects the simulation models that best provide these techniques.

In the third phase of the modeling process, the implementation phase, the model is run and the results are reviewed and analyzed. The validity of the model is determined by demonstrating that the model results represent a reasonable estimate of the actual system behavior. If the model output is not deemed to be sufficiently valid, the input data or modeling technique is modified, and the model is rerun, until the model produces valid results.

9.4.1.2 Parameter Uncertainty and Sensitivity Analysis

The exact value of parameters used in the model are seldom known and frequently vary within defined limits. For instance, an area might be zoned residential with lot sizes less than 0.5 acres and have a Type C Hydrologic Soil Group, so it is given a CN number of 81. In reality, there are some lots that are 0.5 acres, some that are 0.25 acres, and some that are 0.13 acres. The soil also may vary throughout the subwatershed and the Type C soil group is just an approximation for the area. The actual CN number may vary from 70 to 92, but 81 is used as an approximate value for the subwatershed.

A sensitivity analysis is the process of assessing which model input parameters have the greatest effect on the predictions made with the model. The primary purpose of such an analysis is to identify those parameters that should be the focus of the continuing investigations to minimize uncertainty associated with model predictions. Typically, the value of a particular input parameter is changed within some specified range; the model is run again; and new results are generated. The new predicted results are then compared to the original results.

One type of sensitivity analyses uses a standard variational technique that develops a first-order approximation to the variance in the dependant variable (e.g. water surface elevation or flow) to estimate the sensitivity of the dependant variable to changes in a given independent variable (e.g. CN value or watershed area). This can be expressed as:

$$Var\,[Y\,] \approx \sum_{i=1}^{n} \left(\frac{\Delta Y}{\Delta x_i} \right)^2 Var\,[\,x_i\,] \qquad (9.33)$$

where Y = dependent variable
x_i = independent variable
$\Delta Y/\Delta x_i$ = the rate of change in Y with respect to x_i

Equation 9.33 indicates that each of the hydrologic parameters contributes to the overall variance of the predicted dependant variable in proportion to its own variance and the cumulative rates of change of the dependent variable with respect to each of the varied input parameters.

9.4.2 Geographic Information Systems

The function of a GIS in hydrologic modeling is to improve the efficiency and/or quality by reducing the labor intensity of the map manipulations, table look-ups, and repetitious computations required to define input parameters; enable data collection and analysis within variable geographic constructs; and produce more meaningful data outputs in terms of maps, tables, and reports. By reducing the time required to define the input parameters, a larger portion of the project time is available to interpret results and explore alternative design strategies. Although a GIS will allow a hydrologist to be more productive, it cannot replace judgment and experience. Indeed, a well-designed GIS must allow the hydrologist to easily add special conditions to the database and modify pre-programmed procedures when unusual watershed conditions are encountered.

As an illustration of the GIS approach, assume that the SCS procedures described in earlier are to produce subwatershed hydrographs that will be routed through a channel network to generate the peak flow rates and runoff hydrographs required for the design of a bridge. After the input parameters have been defined, the computational tasks will be executed on a computer. The watershed will have to be modeled for both existing and proposed future conditions. After the land cover and hydrologic soil type databases for the jurisdiction have been stored and any needed field data have been obtained from the watershed, a well-designed hydrologic GIS will allow the modeling tasks to be accomplished with the following scenario:

> The hydrologist sits at a desktop workstation and defines a watershed of interest by selecting subwatershed boundaries that were generated automatically from properly scaled digital elevation models (DEMs). The GIS then uses the vector

coordinates of these boundary polygons to access the jurisdiction-wide database and assemble the land cover, land use, and soil data that define the existing conditions in the watershed. The hydrologist inputs any special conditions that have been observed in the field and digitizes the locations of land cover changes that will represent the future condition of the watershed. The digital elevation model is used to identify the location and slopes of the main stream network, minor tributaries, and overland flow planes. Representative channel cross-sections and roughness coefficients for existing and future watershed conditions are defined for specific stream reaches. All the data overlays and other required manipulations are automatically performed to define the parameters and create the software input data. The computer program then produces the existing and proposed condition hydrographs and supporting software provides the array of maps, graphs and tables needed to interpret the analyses.

9.4.2.1 Overview of GIS

Many definitions of geographic information systems technology have evolved, each influenced by the application of interest to its author. A definition that is appropriate for the field of hydrologic modeling is: “A geographic information system is a set of interactive hardware/software tools that integrate and quantify spatially referenced data into quantitative information required for decision-making.”

An example of an application of this definition in hydrology would be to use rainfall, watershed land cover, soil, and topographic data as inputs to a model that provides information in the form of a runoff hydrograph required for a design project.

A GIS integrates data from various sources in disparate scales and differing reference systems and stores this information in a geographically registered database. These data may include a number of layers such as land cover, soil type, and topography. Data can be retrieved, analyzed, and used to produce quantitative information needed to support the decision to be made. The system can be used to reformat geospatial data into formats such as maps, tables, graphs, and text and machine-readable code for input into hydrographic modeling systems that optimize the use and interpretation of information.

Figure 9.20 is a schematic showing the major components of a geographic information system. GIS systems are scalable with reference to data storage, software functionality, and throughput capacity. The current overview of GIS operations in hydrologic modeling concentrates on systems that can be served by a single desktop work station.

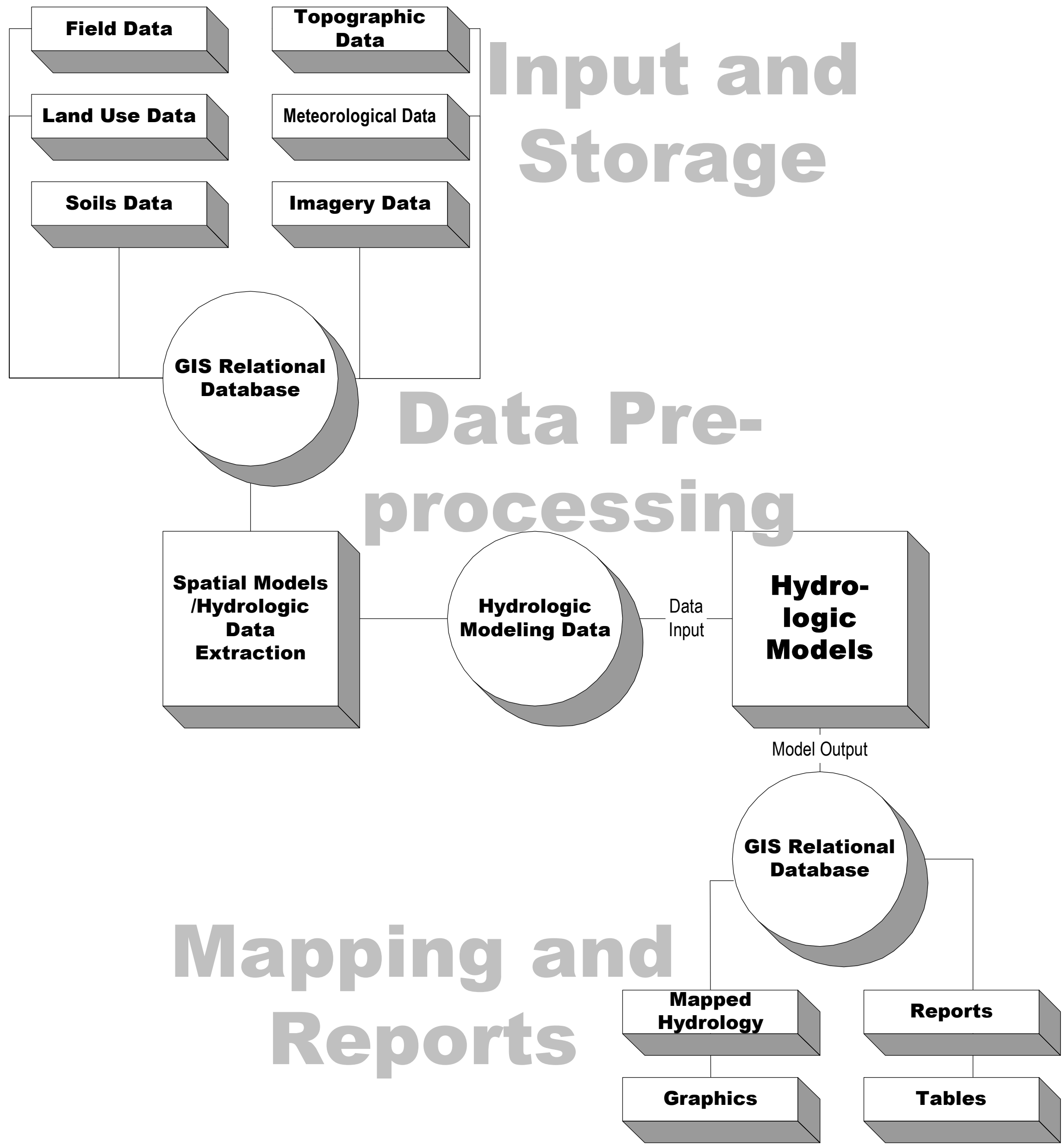


Figure 9.20. Major components of a geographic system integrated with hydrologic modeling.

9.4.2.2 Manual Approach to Input Parameter Generation

This section presents an example to define GIS requirements for hydrologic modeling and to explain some of the concepts of file structure and operation. The example begins by reviewing a manual-based approach to defining model parameters for a small watershed. The objective of the example problem is to use maps and tables to define several parameters that will be inputs to a computer model that generates a design hydrograph. A later section introduces the relevant elements of GIS structure and operations by explaining how the manual operations with the tables and maps are translated into digital procedures, thereby demonstrating the applicability and value of GIS.

In this scenario the hydrologist uses a computer to run a model to generate design hydrographs. Maps and tables are used to define the watershed area, percent of imperviousness, weighted curve number, and the time of concentration. The hydrologist then uses the computer keyboard to type the input parameters into the format required by the model.

The SCS curve number approach is applied in this illustration. A number of widely used hydrologic models use the curve number (CN) to compute the rainfall excess. The CN approach is simple enough to be easily understood. At the same time, the manual overlaying of the spatially distributed land covers and soil types to define a weighted watershed CN are sufficiently difficult to indicate the advantage of computer assistance when the drainage areas are large and diverse. Further, the manual operations with the paper maps and tables listing land cover and soil characteristics provide a good base for understanding the structure and operations with GIS files that are introduced later.

Figure 9.21 illustrates minimum resources needed to define the parameters listed above. Figure 9.21A shows the watershed boundary and the flow network needed to define the area and time of concentration. Figure 9.21B is a plot of the "typical bank-full" stream cross-section that will be used to estimate the velocity in the stream as part of the time of concentration. Figures 9.20C and 9.20D are maps showing the land cover and SCS Hydrologic Soil Groups for an area of 3,660 m (12,000 ft) by 2,130 m (7,000 ft) that surrounds the watershed. The land cover map could have been developed by overlaying a thin paper onto an aerial photograph and drawing polygons around areas having a given land cover. The map of hydrologic soils would have been developed using the county soil maps available from the SCS. Table 9.12 lists symbols that can be used to represent each land cover category that might be found in the vicinity of the watershed, the CN for each soil type and the percent of imperviousness. Table 9.13 is a list of symbols that can be used to represent each of the four hydrologic soil groups.

Historically, a mechanical planimeter or other manual method would be used to determine the area of each land cover category or soil group within each of the polygons of Figures 9.20C and 9.20D. An expedient approach is to overlay a grid as illustrated in Figures 9.21A and 9.21B and assign the symbols of Tables 9.12 and 9.13 to represent the dominant category within each cell. The number of cells in each category is then counted. The smaller the cell size, the closer will be the agreement with the areas obtained using the more accurate, but more time consuming, planimetric approach. Both the planimeter and grid cell processes assume that the soils map, landcover map, and watershed definition are in the same map projection, use the same coordinate system, and are derived from the same scale.

The grid cell representation provides a relatively easy way to develop information required to model the watershed of Figure 9.21A. The grid cell representation of the watershed is illustrated by Figures 9.22A and 9.22B. First, the number of cells within each land cover and soil category inside the watershed are counted and the resulting areas are presented in Tables 9.14 and 9.15. The basin area is 501 hectares (1,240 acres) and the distributions provide an inventory for environmental impact analyses, etc.

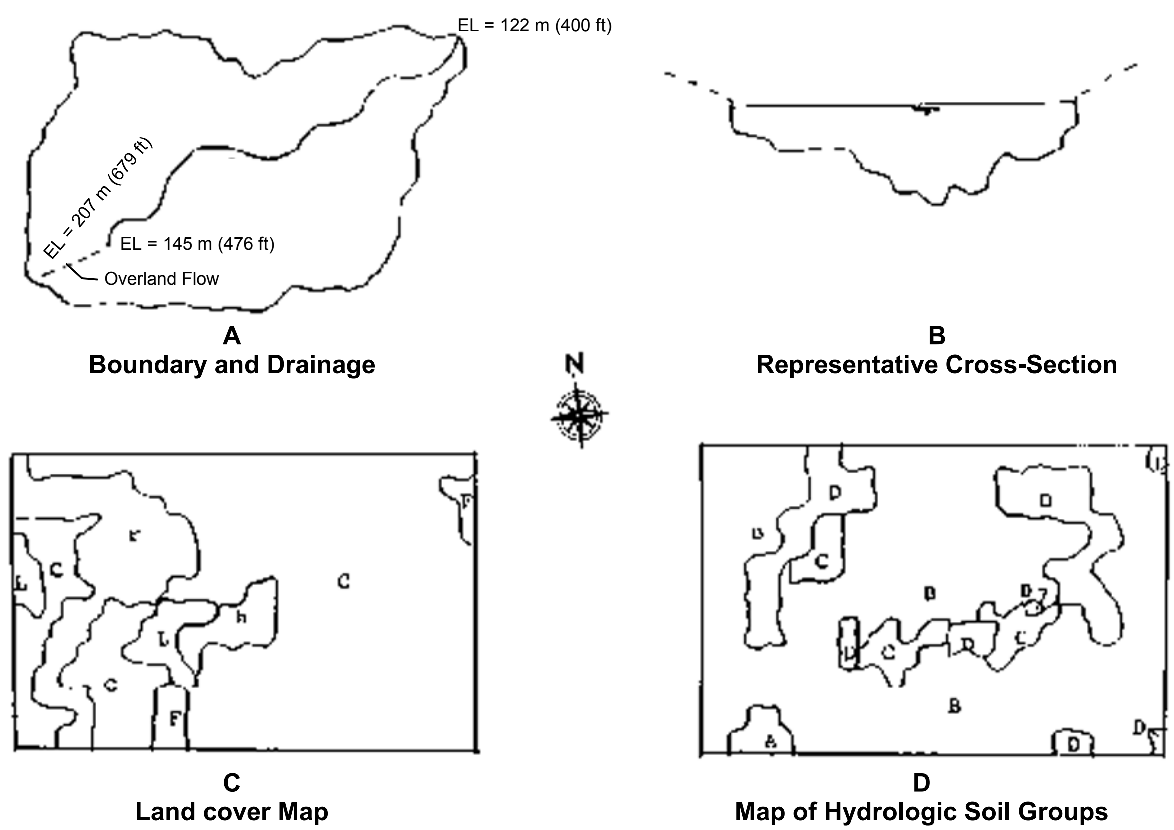


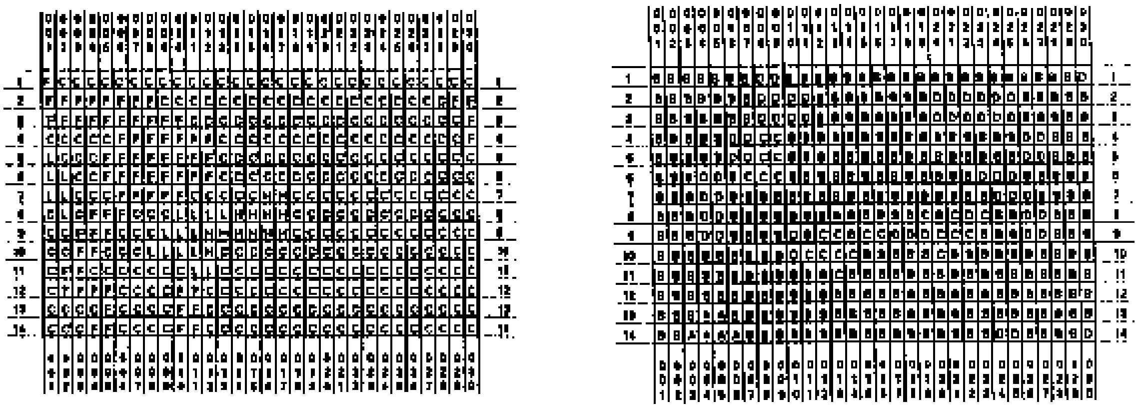
Figure 9.21. Minimum information requirements to run an SCS model

Table 9.12. Characteristics of Land Cover in Area of Interest

				Curve Number				
Line	**Category**	**Sym.**	**%Imp.**	**A**	**B**	**C**	**D**	**Color**
1	RESID(low density)	L	25	54	70	80	85	14
2	RESID(medium density)	M	38	61	75	83	87	12
3	RESID(high density)	H	65	77	85	90	92	6
4	COMM/INDUSTRIAL	A	85	89	92	94	95	4
5	INSTITUTIONAL	I	72	81	88	91	93	5
6	FOREST	F	0	36	60	73	79	2
7	BRUSH	B	0	35	56	70	77	8
8	WATER	W	0	100	100	100	100	1
9	WETLANDS	X	0	100	100	100	100	9
10	BARE SOIL	U	0	77	86	91	94	15
11	CROPLAND	C	0	72	81	88	91	3
12	GRASS	G	0	49	69	79	84	10
13	SURFACE MINING	E	0	77	86	91	94	11
14	CROPLAND-B	@	0	77	86	91	94	7
15	R-30	#	90	90	94	95	97	6
16	RT-12%	%	70	78	88	93	94	5
17	C-2	$	90	88	92	95	96	4

Table 9.13. SCS Hydrologic Soil Groups

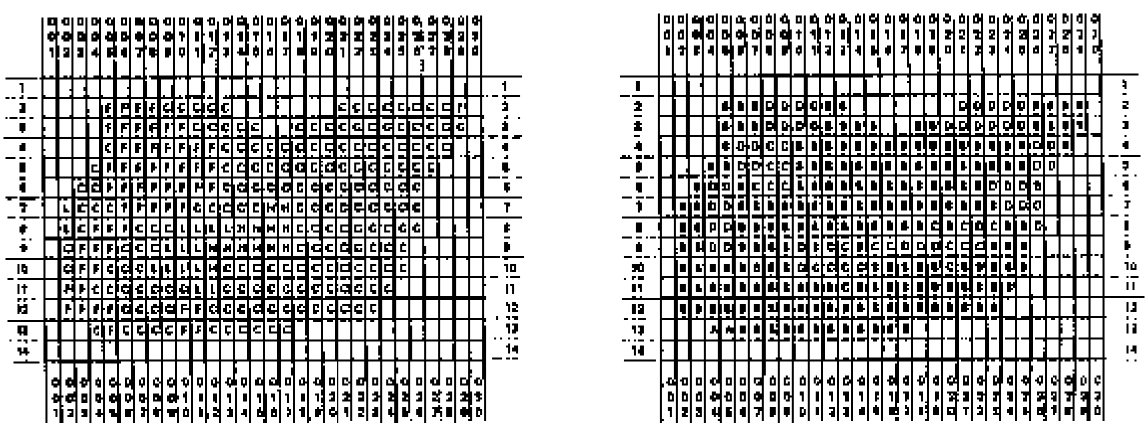
Category	Symbol	Color
Group A	A	14
Group B	B	2
Group C	C	4
Group D	D	8



Land Cover
A

Soils
B

Figure 9.22. Grid cell representation of the spatial distribution of land cover and hydrologic soil groups



Land Cover
A

Soils
B

Figure 9.23. Grid cell representation of land cover and hydrologic soils groups within a watershed

Table 9.14. Summary of Land Cover Distribution in Watershed of Figure 9.21

Symbol	Category	Number of Cells	Area (ha)	Area (ac)	Percent
L	RESID (low density)	15	27.82	68.7	5.56
H	RESID (high density)	13	24.11	59.6	4.81
F	FOREST	58	107.56	265.8	21.48
C	CROPLAND	184	341.20	843.1	68.15
Total		270	500.69	1237.2	100.00

Table 9.15. Summary of Hydrologic Soil Group Distribution in Watershed of Figure 9.21

Sym.	Category	Number of Cells	Area (ha)	Area (ac)	Percent
A	GROUP A	2	3.71	9.2	0.74
B	GROUP B	197	365.32	902.7	72.96
C	GROUP C	23	42.65	105.4	8.52
D	GROUP D	48	89.01	219.9	17.78
	Total	270	500.69	1237.2	100.00

The distribution of the cells shown in Figures 9.22A and 9.22B are used in conjunction with Tables 9.12 and 9.13 to define the composite runoff curve numbers required by the SCS models. For example, Table 9.12 shows that the "F" representing the dominant land cover in cell (8,2) of Figure 9.23A is "Forest". Figure 9.23B shows the corresponding soil cell to be in the D hydrologic group. Table 9.12 is then used to show that cell (8,2), an area of "Forest" on a "D Hydrologic Soil", has a curve number of 79. This overlay/ table look-up process is extended for all cells within the boundary to estimate an average or "weighted" curve number for the watershed. Table 9.16 illustrates one approach that can be used to manage the cell counting process and Table 9.17 shows the computations to define the average curve number and percent of imperviousness.

Table 9.16. Example of Type of Tabulation Used to Define Cell Counts for Curve Number Computation

Land Cover Category	Cells in Soil Group				Total Cells
	A	B	C	D	
RESID(low density)	0	12	1	2	15
RESID(high density)	0	7	5	1	13
FOREST	1	37	7	13	58
CROPLAND	1	141	10	32	184
Total	2	197	23	48	270

Table 9.17. Example of Weighted Curve Number Computation

Land Cover Category	**Cells x Curve Number** A B C D	**Product**
RESID(low density)	12(70) + 1(80) + 2(85) =	1,090
RESID(high density)	7(85) + 5(90) + 1(92) =	1,137
FOREST	1(36) + 37(60) + 7(73) + 13(79) =	3,794
CROPLAND	1(72) + 141(81) + 10(88) + 32(91) =	15,285
Total		21,306

Weighted Curve Number = 21,306/270 = 79
Percent Imperviousness 15(25) + 13(38) = 869/270 = 3.2%

The first step in determining the time of concentration is to consider the main stream. The watershed time of concentration is the sum of travel times through the stream network and overland flow. These equations require that the hydrologist have information on the slope and cross-sectional characteristics of the main stream and overland flow, as is the case with most physically-based models.

The next step is to arrange the watershed area, weighted curve number, time of concentration, and the precipitation of interest into the required format for keyboard input to the model. Even this task can be frustrating because the format requirements of many models are quite cumbersome.

These are the steps used to model the watershed under existing conditions. With increasing frequency, the hydrologist must develop hydrographs for some proposed condition where no streamflow data exist or where land covers have changed on parts of the watershed and all or some of the drainage network has been modified. If proposed conditions have to be modeled, the changes would be made on Figures 9.20A, 9.20B, 9.20C, and 9.22A, and all the above steps repeated with little gain in efficiency over the existing condition analysis.

9.4.2.3 Translation of Manual Approach into GIS Procedures

The map-based steps used to define the area, curve number, and time of concentration are tedious even for this small watershed. Through GIS technologies, the manuaul steps described previously can be translated into equivalent digital procedures that can be executed in a fraction of the time required by conventional approaches. Some of the pertinent GIS concepts will now be explored by translating the map-based approaches described above into generic GIS operations.

In the example, the spatial distributions of the land cover and soil databases were represented and analyzed as the arrays of grid cells shown in Figure 9.22A and 9.22B. When this grid cell representation is translated into a digital format for use in a GIS, it is termed a "raster data structure" or a raster file. Data can be entered into the GIS system using any number of data entry techniques in which the symbol is translated into a digital number for each cell. Each symbol would represent the dominant land cover or soil category in the rectangular area located at the indicated column/row. If data are provided by reputable distributors, e.g., USGS and NRCS, the data will most commonly be georeferenced. Map coordinates, physical cell size, map projection, map coordinate system, and levels of precision are recorded and transmitted as Metadata with each data set. This process enables the hydrologist to store information that crosses jurisdictional lines and extract data based upon a geographical description of the watershed.

In performing the tasks involving Figures 9.22A and 9.22B, the first step was to note the symbol for each cell. Tables 9.12 and 9.13 were then used to determine the category the cell represented and to assign percent imperviousness and a curve number. In a GIS, the digital equivalents of Tables 9.12 and 9.13 are called "attribute tables" and are related to the digital value of each cell. As in the manual example, attribute tables assign properties to digital values in the raster database. For example, if the symbol "H" is accessed in the land cover raster database, the attribute file of the form of Table 9.12 is accessed to identify the land use in the cell as "RESID(high density)" with an imperviousness of 65 percent.

In the context of this example, the hydrologic GIS is designed to duplicate the steps that would have traditionally been performed manually, but with much more speed, reproducibility, and quality control. One of the major capabilities of a GIS system is the ability to model spatially related data to perform data extraction (spatial queries) and data analysis by applying mathematical operations to data. A simple example would be, column 8 on row 2 of the raster equivalent of Figure 9.23A is overlaid onto the corresponding location on the equivalent file of Figure 9.23B. The match is "F" land cover on a "D" soil. The attribute file representing Table 9.12 is then accessed to assign a CN=79 to the cell in the same manner as described in the manual approach.

It is important to note that the raster format is only one approach to representing the spatial distribution of the land cover and soil categories in Figures 9.20C and 9.20D. Most GIS systems integrate raster and vector (line and point data) into unified systems. Neither format is inherently “better” or “more powerful” than the other. Current GIS systems use the two formats to complement one another to expedite processing and maximize quality.

In the manual approach, the watershed boundary was drawn and visual inspection selected the cells that were inside. In the GIS environment, a polygon representing the boundary is created in geographic space. The boundary is used to query and extract all coincident data available within the selected polygon.

In the manual approach, the lengths of the stream and overland flow plane were measured on Figure 9.21A. A hydrologic GIS delineates stream reaches and stores lengths of streams and overland flow segments based upon digital topographic relationships. The elevations required for the slopes are extracted from digital elevation models.

After these inputs are provided by the hydrologist, the GIS software places the watershed area, weighted curve number, and time of concentration into a file formatted for entry into the hydrologic model being supported. Similar steps are followed by GIS software when the hydrologist selects methods other than the SCS curve number approach.

Data used throughout hydrologic analysis can also be used to support parallel issues such as environmental impact studies, sediment control programs, economic impacts, etc.

9.4.2.4 GIS Requirements for the Modeling of a Complex Watershed

A more complex case study is considered in this section. Figure 9.21A is Subwatershed 8 in the larger basin represented by Figure 9.24A. The drainage area of the watershed shown in Figure 9.24A is 131 km^2 (50.6 mi^2). In a watershed of this size, travel time, nonhomogeneous conditions, and floodplain storage in the stream network will have a major impact on the hydrograph at the watershed outlet. The stream system is simulated by combining and routing the hydrographs from each subwatershed through the dendritic network shown in Figure 9.24B using one of the routing techniques presented in an earlier chapter. As stated earlier, the computational intensity of these tasks lead hydrologists to rely on computer programs. The use of GIS to support the hydrologic modeling of complex watersheds, such as the example in this section, is discussed by Ragan (1991).

As an additional requirement, assume that the watershed of Figure 9.24 has to be modeled for both the existing and proposed land cover distributions shown in Figure 9.25. When an organization is using a GIS to support the modeling of watersheds, the land cover/curve number attribute table is probably more like Table 5.4 than Table 9.12. Two situations typically occur in hydrologic modeling. First, field investigations may reveal that there are areas in the watershed that are different from any of the categories in the attribute table that has been prepared for general use. Second, modeling future conditions is frequently based on local zoning designations rather than the names of land cover categories that appear in the attribute files. Thus, the GIS must allow the attribute files, digital equivalents of Table 9.12 or Table 5.4, to be easily edited so that new land cover and zoning categories can be added or deleted for use on a particular watershed. In the case illustrated by Figures 9.25A and 9.25B, a "CROPLAND-B" has been to be added to improve the representation of the existing land cover, and three zoning designations have been added to describe the anticipated future development.

In defining the input parameters needed to model the watershed of Figure 9.24, the steps described in Section 9.4.2.3 are followed to determine the drainage area, curve number, and time of concentration for each of the 13 subwatersheds for both existing and proposed conditions. To accomplish this, each specific data layer can be initially input in the form of Figure 9.23 to reflect current conditions and then edited to reflect anticipated changes. For example, after developing the existing condition model, the area to be changed to "CROPLAND-B" for future conditions is edited. All other areas that are being rezoned for future development are also edited. A second database that stores the land cover distribution of Figure 9.25B is then generated to support the definition of the watershed under future conditions.

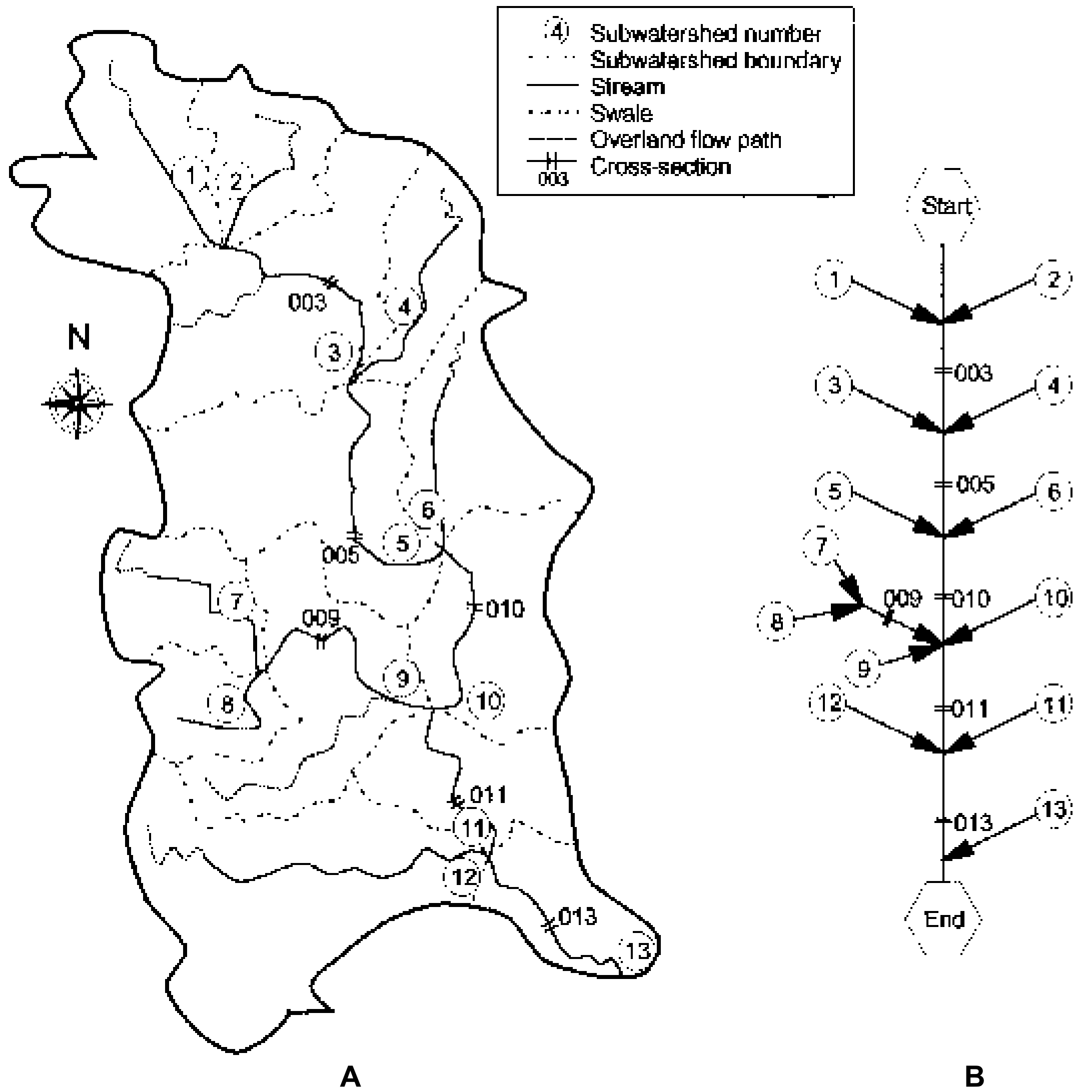


Figure 9.24. Distribution of subwatersheds and network representation of a complex river basin

The next step is for the GIS to assist in setting up the data that define the stream network that will control the hydrograph combining and routing. The stream junctions will correspond to the outlet locations of the subwatersheds and represent points where subwatershed hydrographs will be combined before being routed.

Bank-full stream cross-sections and a Manning roughness coefficient are used to determine the velocities needed for calculating the time of concentration and routing parameters. Some flood routing models require complete cross-sections along with main channel, left overbank, and right overbank roughness coefficients. The GIS can be configured to accept the cross-sections from survey notes, plots, or through interpolation along digitized contours. In the example of

Figure 9.24, some routing techniques require the cross-section data to be translated into some form of stage-discharge tables for cross-sections 003, 005, 009, 010, 011, and 013.

When modeling a watershed of the complexity of Figure 9.24, the computation and merging of the subwatershed hydrographs and routing through the stream network usually involves the use of computer programs. The input file required by the model must not only define the model parameters, but also, the linking and routing processes illustrated by Figure 9.24B. The well-designed GIS should incorporate "network analysis" that uses the digitized stream segments and junctions to automatically set up the input file that will cause the model to be executed in accordance with the watershed schematic of Figure 9.24B.

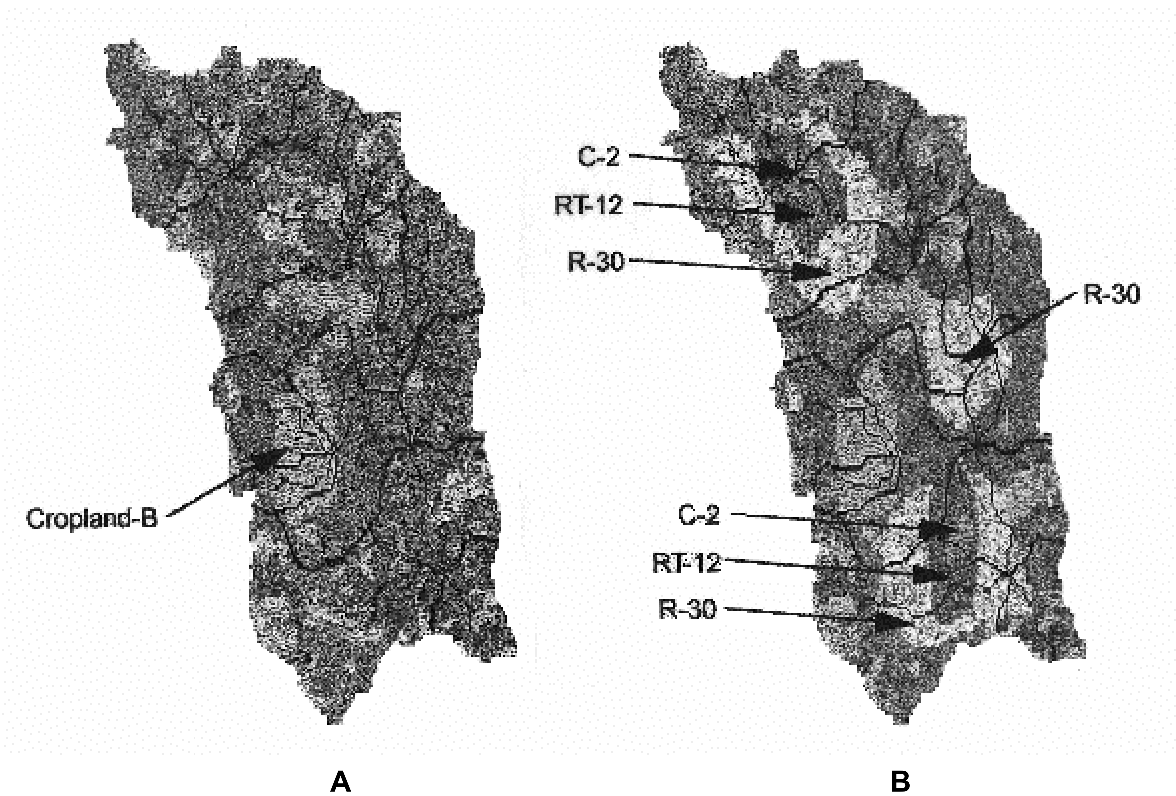


Figure 9.25. Existing and ultimate development land cover distributions in a complex watershed

9.4.3 GIS Implementation Issues

9.4.3.1 Storage and Resolution

A state or county highway department may conduct modeling studies such as those described earlier many times during a year. One project may be in one part of the jurisdiction while the next will be in another area. If the watershed sizes are above some minimum value and the objectives of the modeling efforts are similar from one project to the next, the optimal approach is to develop a jurisdiction-wide database that will be maintained and immediately available on the hard disk of the workstation. In a traditional hard-copy map-based approach, the hydrologist goes to storage cabinets to obtain the topographic maps, aerial photos or land cover maps and the soil maps. With the data available from sources available on the system network, the hydrologist would simply use the mouse to point to the data to be retrieved.

The hydrologic database can be very large, especially if it is to support a GIS that will be used anywhere in a state. Even with the efficiencies of today's workstations, a large database must be properly structured if it is to be quickly accessed and easily maintained. Network access to data from local, state, federal, and private sources have been enabled through the internet. Baseline data collected for one hydrologic study may have applicability to another hydrologic model. Therefore, sound database and network design are critical to being able to store, retrieve, and update geospatial data sets.

Data sharing among state, local, and federal agencies is active and growing as of this writing. It is not necessary for the individual highway department to create and store all of the data required for hydrologic studies, only to request access to the data stores of other agencies. It is critical that the hydrologist understand the nature or level of detail stored in each dataset. Modeling very small watersheds with a high quality model can require the location of each building, road, parking lot and storm sewer along with a detailed description of the soil distribution. The hydrologist must match the scale, level of detail, and currentness of the data to the scale of the hydrologic study.

Modeling watersheds when their areas are larger than around 60 ha (150 ac) can be accomplished with the more general land cover categories such as those in Table 9.12 stored as an array of 1.86 ha (4.60 ac) cells in a raster format (Ragan, 1991). Four-acre cells are considered to be quite coarse with modern systems. National Land Cover Data (USGS) are available in 30m cell sizes, 0.2 ac per cell. Thus, a practical approach is to select a lower limit on the size of a watershed to be modeled and build a jurisdiction-wide, on-line database to support that task. The GIS is then designed to allow the hydrologist to develop optimal resolution. Local databases to support the modeling on special projects, such as the watershed illustrated by Figure 9.26, can be developed on a case by case basis.

It is not necessary to store land cover in cells that represent an area of 1 ha (2.5 ac) if the area is homogeneous and the data can be stored in 5 ha (12.5 ac) cells without changing a parameter beyond some acceptable, predetermined limit. In general, relatively large cells can be used to represent the spatial distribution of land cover in the agricultural fields of the great plains, but, much smaller cells would have to be used to adequately describe a suburban or urban area. The GIS system can provide accurate tabular results to be included in the hydrologic models from either scenario. Each layer can be independent of other layers, e.g., county level soils data can be used in its native level of precision along with statewide land use data. Sensitivity studies would normally be conducted to gain insight into the consequences of changing the size of the data cell.

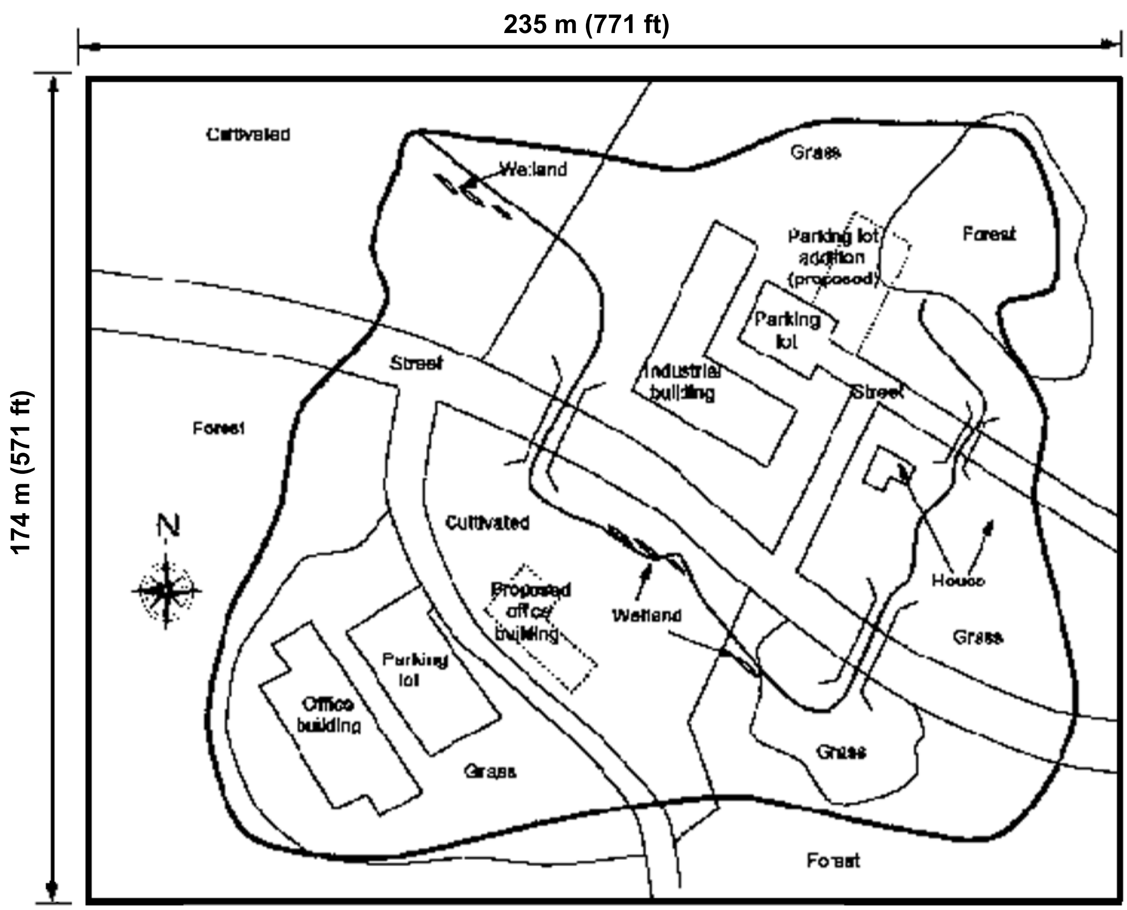
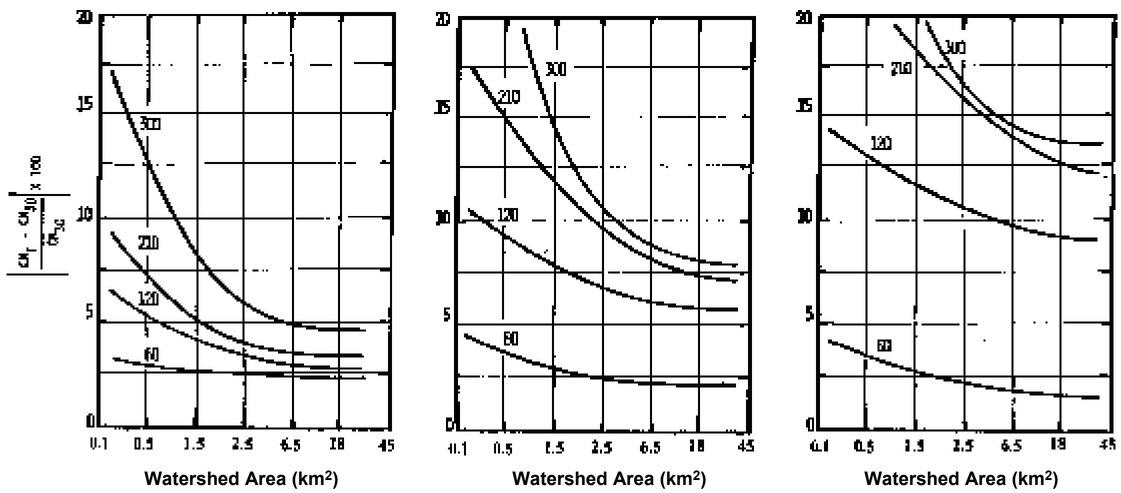


Figure 9.26. Example of a detailed land cover distribution required for the modeling of a very small watershed

Figure 9.27 indicates how a sensitivity study could be conducted. The ordinate in each of the plots is the percent change in the estimate of CN with the most detailed resolution (30 m resolution) as the reference point. Each of the curves in each plot is labled with the resolution of the comparative data, e.g. 60 m and 120 m. The alternative distributions represent three different land/soil complexes. Though not described here, the land cover distributions I, II, and III increase in complexity. The problem is to define how much a curve number is changed as the size of the data cells is increased from 30 to 60 to 120 to 210 to 300 m (100 to 200 to 400 to 700 to 1,000 ft). If the study area is quite large and the time and resources are restrictive, it may be beneficial to use the largest possible cell size to minimize database development and operation costs. If the database covers an area having a land cover distribution similar to Distribution I, then the data cell could be increased from 30 m to 210 m (100 ft to 700 ft) and only change the curve number by 5 percent for a watershed of 1.5 km² (0.6 mi²). If land cover distribution II were involved, 210-m (700-ft) data cells would give curve numbers for a 1.5 km² (0.6 mi²) watershed that differed from that obtained with a 30-meter cell by approximately 12 percent. For Distribution III, the difference would be about 23 percent.



Distribution I **Distribution II** **Distribution III**

Figure 9.27. Change in estimated curve number for increasing grid cell size for three land cover soil complexes

If the data are to be stored in a vector rather than a raster format, a similar sensitivity study is required. Instead of testing for the minimum cell size, the quest is for the "minimum mapping unit", the smallest polygon storing data. The minimum mapping unit is important because many county land cover maps and government-distributed databases are specified in terms of this unit. The minimum mapping unit is identified within the metadata of each dataset.

The hydrologist needs the results of the sensitivity studies to make a decision that balances the requirements of the parameter estimates with the economics of database development and processing. If the modeling objectives can be met with a 120-m (400-ft) database, the hydrologist will be developing and working with one-sixteenth the data that would be involved with a 30-m (100-ft) database.

9.4.3.2 Sources of Digital Format Geographic Data

If the hydrologist chooses to develop a GIS database to meet the requirements of hydrologic modeling, there are a number of digital format data products that can be used. The Federal Government can be an excellent source of digital format data that can be integrated into a GIS. This section discusses some of the most widely used digital format data.

Generally, the most expedient approach to the development of a GIS database is to obtain data that are already in a map referenced digital format. Thus, a first step in the development of a database should be to contact agencies in the region to determine where hydrologic data of the appropriate level of detail can be obtained. The first search mechanism can be the internet. Most local, state, and federal agencies distribute hydrologic data in transportable GIS format.

The hydrologist should be aware of differing map projection, e.g., Albers Equal Area, Lambert Conformal Conic; coordinate systems, e.g., State Plane Coordinates, UTM; and datum references, e.g., NAD27, NAD83, GRS80. Failure to identify map projection issues can result in significant data offset and misalignment. Most modern GIS systems perform "projection on the fly" which will properly align data automatically. The hydrologist may choose to reproject all GIS data into a standard projection and coordinate system to minimize the chance of misalignment errors.

The USGS distributes digital land cover and land use at scales of 1:100,000 or 1:250,000 and 30 m resolution National Land Cover Data (NLCD). There are 37 land use and land cover categories stored as polygons as small as 4 ha (10 ac) - the minimum mapping unit for digital land cover land use. Most of the land cover information for the US was defined in the mid-1970s. The metadata for each dataset identifies the date of acquisition. The files also contain political boundaries, hydrologic units, and federal/state land ownership information. There are 21 land cover classes in NLCD. These data were created from satellite imagery in the early to mid 1990s and are updated periodically.

Satellite imagery and aerial photography is an important resource for the development of a land cover database. The satellite industry is changing rapidly, providing higher resolution (0.6 m panchromatic data at this writing) with more frequent re-visit times. Data costs are highly variable depending upon the level of detail requested and the timeliness of the data. Imagery from satellites are available in either photographic or digital formats. Photographic reproductions of satellite imagery are often a realistic approach to define land cover distributions. If digital format satellite imagery is to be the source, the hydrologist must ensure that the supplier of the land cover data is experienced and well-equipped for image processing. The hydrologist needs to ensure that any imagery requested match the map coordinates and projection system used for the rest of the study, especially if photo products are acquired.

The USGS also distributes Digital Ortho Quarter Quadrangle (DOQQ) data either through the State or through the NRCS. DOQQs are orthogonal to the surface of the earth and are available in color or B&W in 1 m spatial resolution. The metadata provide the creation date of a DOQQ.

The USGS digital elevation model (DEM) data, an array of regularly spaced elevations, has found increasing applications in hydrologic modeling. DEM data in 7.5-minute units are spaced at 30 m (100 ft), and in some cases 10 m (30 ft) spacing, while the 1-degree units are spaced at 3 arc seconds. Care must be exercised when using the watershed definition capabilities of a GIS system using a DEM. The accuracy of automatic watershed delineation with a DEM decays significantly as the number of elevation points inside a watershed boundary decrease. Also, the hydrologist must be sure that the level of precision of the DEM is sufficient for the watershed being studied and that the elevations are current. If the study area is in a rapidly developing area, the DEM may not reflect current conditions. The metadata provide the creation date of a DEM.

The USGS also distributes Digital Raster Graphics (DRG), which are georeferenced scanned quad maps. These maps are available for 1:24,000, 1:100,000, 1:250,000, and 1:1,000,000 scale maps for the entire US. These data can be used as standard map backdrops to ensure the spatial accuracy of hydrologic studies.

The USGS digital line graph (DLG) data are digital representations of the cartographic information on topographic quadrangle maps. DLG data are available in nine categories that provide digital representations of features such as streams, watershed boundaries, roads, vegetative cover, buildings, and transportation networks. The metadata provide the creation date and the level of precision of DLG data.

The National Wildlife Service is the primary producer and distributor for National Wetland Inventory (NWI) data. The data can be obtained in either 1:24,000 or 1:100,000 scale for points, linear features, and polygons. The metadata provide the creation date of NWI data.

The NRCS of the U.S. Department of Agriculture is the primary source of data concerning soils. The NRCS has developed computerized databases to integrate soil map information with other data in geographic information systems for most of the US. These digital format databases are being developed at three levels of detail with the Soil Survey Geographic Database (SSURGO) being the most appropriate for hydrologic modeling associated with highway drainage structures. SSURGO is not available for all counties in the United States. The availability of SSURGO and other levels of NRCS digital soil data within a region of interest can be determined by contacting the respective NRCS state office or the NRCS internet distribution site for a status map of SSURGO data. STATSGO soil association data are available for virtually all of the US in digital format. These data may be appropriate for large watershed studies or if SSURGO data are unavailable for a specific region.

The U.S. Bureau of the Census provides several digital products that can be of value in a hydrologic GIS. The TIGER/Line files (Topological Integrated Geographically Encoding and Referencing) can be used to plot streams and roads. The TIGER files used in conjunction with the Bureau's socio-economic data can provide important information for hydrologic modeling on urbanizing watersheds. The hydrologist should be aware that the positional accuracy of pre-2004 TIGER data is derived from 1:250,000 scale USGS DLG data. After 2004, the positional data will be derived from 1:24,000 DLG data, a much more precise dataset. Logical accuracy, e.g., the population of a town, is quite good, however.

FEMA distributes some floodplain maps in digital format. FEMA is also embarking on several programs for digitizing floodplain maps around the country.

Nearly all of the federal digital format data products, such as the USGS land use/land cover, can be downloaded through the Internet. There are also a number of "value added" companies that sell geographic databases that cover a state, county, or other political unit. The nucleus of these databases is typically one of the federal products described in this section. The company may add data to better reflect existing local conditions, reformat files for easier use, provide software to improve access to the data, and provide technical assistance.

State, local, and private sources may have higher level of detail data available for low or no costs. Highly detailed elevation models are sometimes derived from aerial photography, radar (IFSAR), or Light Detection and Ranging (LIDAR) data. These data may be obtained on contract for FEMA flood studies or other funded research. Several vendors provide these services. Small watersheds can be defined within ± 2” of actual elevations using these technologies.

9.4.3.3 Digitizing Paper Format Data Sources

The plethora of readily available digital data reduces the necessity of using paper format maps. However, if the source of some specialized data is paper format, there are two approaches to translating the data into a digital format.

Areas, lines, and points can be translated into digital format with either a digitizing tablet or an optical scanner. The digitizing tablet has been the "standard" for many years. In this approach, the technician traces lines with the cursor. Software then translates the digitizer inputs into the required GIS formats. In a scanner approach, the paper product is placed on a glass stage and a light source transfers the image into a computer workstation. A technician interacts through screen prompts with software that translates the image into the GIS formats. The scanner is generally faster than the digitizing table and less subject to operator error. A discussion of the issues that defined these two approaches is presented by Ragan (1991).

The county soil maps published by the U.S. Department of Agriculture NRCS are available in paper format source of soil data if SSURGO data are not available. These maps can usually be obtained from the state offices of the NRCS.

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