



INTERNATIONAL
EDITION



Hydrology and Floodplain Analysis

FIFTH EDITION

Philip B. Bedient • Wayne C. Huber • Baxter E. Vieux

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FIFTH EDITION

Hydrology and Floodplain Analysis

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Dedication

*To Cindy, Eric, and Courtney,
to my parents for their guidance, and
to my teachers, and to all my students
over the past 35 years.*

P.B.B.

To My Family

W.C.H.

*To my wife, Jean, and to our children
William, Ellen, Laura, Anne, and Kimberly,
and to my parents*

B.E.V.

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PREFACE

The field of hydrology is of fundamental importance to civil and environmental engineers, hydrogeologists, and other earth scientists because of the environmental significance of water supply, major floods and droughts and their management, drainage and urban stormwater issues, floodplain management, and water quality impacts. In recent years, hurricanes and storm surge have caused significant urban disasters, especially along coastal areas where urban development has expanded rapidly. This text was written to address the computational emphasis of modern hydrology at an undergraduate or graduate level, and to provide a balanced approach to important applications in hydrologic engineering and science.

Increasing use and sophistication of personal computers has revolutionized the daily practice of hydrology. The Internet has revolutionized the accessibility of hydrologic data and computer models. The impact of transferring online data from governmental and scientific sources to the practicing hydrologist or student has been phenomenal. Hydrologic data, digital terrain models, and mapping software linked with hydrologic modeling have allowed complex problems to be solved. Geographical information systems (GIS) and radar rainfall have greatly improved our ability to predict hydrologic response using distributed information. A number of major improvements have been made to existing hydrologic models, such as HEC and SWMM, and advances in distributed hydrologic modeling such as Vflo. A list of important websites and links currently used routinely in hydrology is contained in Appendix E and can be found at the textbook website: *hydrology.rice.edu*.

THE EVOLUTION OF HYDROLOGY

-
- **Chapter 1:** Extensive updates and changes, and completely rewritten with a more introductory coverage of topics, including simple rainfall runoff and hydrograph analysis with new examples and completely revised homework problems.
 - **Chapter 2:** Coverage of the unit hydrograph and includes hydrologic losses such as evaporation and infiltration. New examples have been added on Green and Ampt infiltration, as well as many additional homework problems.
 - **Chapter 4:** Expanded coverage and new examples on flood routing. Many of the older numerical schemes have been simplified or removed, and new homework problems have been added to streamline the chapter.
 - **Chapter 5:** Presents updated methods for simulating rainfall and runoff, flood hydrograph prediction, and flood control options in a

NEW TO THIS EDITION

watershed. New models are highlighted with new examples and a new detailed case study, including flood control alternatives. All new homework problems have been written as well.

- **Chapter 10:** Describes the application of GIS and geospatial information in hydrologic analysis and formulation of hydrologic models. An updated list of available GIS and related software is provided.
- **Chapter 11:** Major advances in detecting accurate and representative rainfall with radar and rain gages are presented. The application of radar rainfall as distributed model input is described. Details on distributed hydrologic modeling are presented for a case study, for which sample datasets are available in Appendix E.
- **Chapter 12:** Reviews the new emerging trends in flood control methods and floodplain management, a vital topic given the recent storm events in the United States, such as Hurricane Irene in the northeast. A detailed discussion of massive hurricane impacts along the Gulf Coast, including Hurricane Katrina and Ike, two of the most damaging hurricanes in history.
- **Chapter 13:** Brand new and includes emphasis on the San Antonio River and Edwards Aquifer System in Central Texas, the Colorado River Basin system, and the Thames River in England. The chapter also briefly highlights international water issues in China and South-east Asia. The chapter ends with a discussion of the changing global condition and the need for sustainable interaction between human and natural systems.

**ORGANIZATION
OF THE 5TH
EDITION**

The 5th edition of the text is divided into three main sections. The first section, consisting of the first four chapters, covers traditional topics in hydrology related to the water balance such as: (1) hydrologic principles, hydrologic cycle, and measurement techniques, (2) hydrologic analysis using hydrographs for rainfall-runoff, (3) statistical and flood frequency analysis, and (4) hydrologic and hydraulic flood routing methods. The second major section, Chapters 5 through 9, is designed to apply hydrologic theory and available hydrologic modeling techniques to several areas of engineering hydrology and design: watershed analysis, floodplain delineation, ground water, and urban stormwater. The latest methods and computer models are emphasized in enough detail for practical use, and updated examples and new case studies are provided.

Chapter 5 on Hydrologic Simulation Models has been completely redone and updated to include the latest model versions as highlighted below in the next section. Chapter 6, Urban Hydrology, presents methods and reviews available computer models for pipe and open channel storm drainage systems. The Storm Water Management Model (SWMM5) is highlighted as the most comprehensive urban runoff model available today. Chapter 7, Floodplain Hydraulics, first reviews concepts from open channel flow and includes uniform flow and critical flow coverage, as required to understand water surface

profile computations. The HEC-RAS model is described in detail with a case study demonstrating the power of the model to evaluate natural floodplains. Chapter 8 presents ground water hydrology as a stand-alone chapter, including flow in porous media, aquifer properties, well mechanics, and computer applications. Governing equations of flow are derived and applied to a number of ground water problems, including both steady-state and transient analyses. Chapter 9, written by James Thompson, is a comprehensive chapter on design applications in hydrology. It addresses design rainfall, small watershed design, hydraulic design, detention pond design, detailed culvert design, and floodplain mitigation design issues. It is one of the most comprehensive design chapters ever written and presents both the theory and method behind modern hydrologic and hydraulic design techniques.

In the final section of the 5th edition, three chapters (10, 11, 12) guide the user to the next generation of hydrologic computation, watershed evaluation, and discussion of important severe storm impacts and flood management. No other textbook presents this type of material all in one place. Chapter 13, a new chapter for the 5th edition, presents details of major water resource projects around the world.

Chapter 10 presents the use of geographic information systems (GIS) and digital elevation models (DEMs) as important tools for watershed analysis, hydrologic modeling, and modern floodplain delineation. Many useful hydrologic datasets and software are now widely available in high-resolution digital form on the web (see Appendix E). Chapter 11 depicts some of the latest technology on the use of NEXRAD radar data to estimate rainfall intensities over watershed areas. Radar rainfall has greatly improved our ability to predict rainfall patterns over a watershed and offers real advantages for hydrologic flood alert systems. Distributed hydrologic modeling is introduced that capitalizes on advances in radar rainfall detection.

The 5th edition of the text should provide the engineering or science student with all the necessary theory to understand principles of hydrology, hydrologic modeling, floodplain hydraulics and analysis, and water resources in the modern digital world. The student or practicing engineer should find the book a useful reference for hydrologic methods, current models, design examples, and extensively documented case studies. In addition, simple calculations and spreadsheet examples are utilized and highlighted in numerous places in the 5th edition, which contains over 80 worked examples, over 220 homework problems, and 6 major case studies.

The World Wide Web offers many opportunities for access to regional data with minimal cost and effort (see Appendix E), but the U.S. Geological Survey, National Weather Service, National Resources Conservation Service, U.S. Army Corps of Engineers, and other state and local agencies should be emphasized for students as likely sources of regional hydrologic data.

The text includes a Companion website *hydrology.rice.edu* where updates and information can be found regarding the 5th edition. Dr. Bedient maintains the site that contains selected problems, example datasets,

**AUDIENCE AND
AVAILABLE
RESOURCES**

simple Excel programs, and dozens of PowerPoint presentations that are available to any instructors and students in the course at no charge. These resources are designed to improve the teaching of a hydrology course at either the undergraduate or graduate level. A detailed solutions manual with updated problems is available for the 5th edition. The solutions manual for the International Edition can be accessed at www.pearsoninternational editions.com/bedient.

ACKNOWLEDGMENTS

The textbook was developed over a period of 25 years, beginning in 1982, from original course notes in a class in Hydrology and Watershed Analysis. During the many years of interaction with colleagues and students, the book evolved into its present form with emphasis on simple examples, clear explanations, and modern computational methods. The 5th edition includes a new Chapter 13 written by Antonia Sebastian and several existing chapters with significant updates such as Chapter 5 (Zheng Fang and George Doubleday) and Chapter 12 (Jude Benavides and Heather Winter).

We are particularly indebted to the following individuals for their careful review of the draft manuscript and for numerous suggestions and comments: Michael Boufadel, Temple University; Stephanie Moret, Louisiana State University; Rao S. Govindaraju, Purdue University; William R. Wise, the University of Florida; and Francisco Olivera, Texas A&M University.

Holly Stark, Executive Editor at Prentice Hall, was instrumental in guiding the significant changes for this 5th edition; the authors thank all the professionals at Prentice Hall for their efforts on our behalf.

A successful textbook always represents a team effort, and the team at Rice University has been excellent in their continuing support and attention to detail. Antonia Sebastian contributed by organizing and writing Chapter 13. Dr. Zheng Fang and George Doubleday contributed to Chapter 5, and Heather Winter added sections and review for Chapter 12. Special thanks are due to students Tatyana Luttschlager and Roni Deitz at Rice University for their technical skills in reviewing text and figures, and for providing valuable input to examples and homework problems. We would like to thank Erin Baker for helping to organize text, figures, pictures, and permissions for the 5th edition, as well as Mr. Rik Hovinga at Rice University for excellent graphics design, picture selection, and the design of the image that appears on the front cover. We would also like to thank Rice University students Johan Loveless, Alexandra Rasch, and Mike Burcham who assisted greatly in solving examples and developing and checking new homework problems for the 5th edition. Maria Corcuera was also instrumental in helping to organize text and figures for the 5th edition.

Philip B. Bedient, *Rice University*

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The publishers wish to thank S. Saravanan of the National Institute of Technology, Tiruchirappalli, for reviewing the content of the International Edition.

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Philip B. Bedient is the Herman Brown Professor of Engineering, with the Department of Civil and Environmental Engineering, Rice University, Houston, TX. He received the PhD degree in environmental engineering sciences from the University of Florida. He is a registered professional engineer and teaches and performs research in surface hydrology, modeling, and flood prediction systems, and ground water hydrology. He has directed over 55 research projects over the past 36 years, and has written over 180 journal articles and conference proceedings over that time. He has also written four textbooks in the area of surface and groundwater hydrology. He received the Shell Distinguished Chair in environmental science (1988–92), the C.V. Theis Award in 2007, and he was elected Fellow of ASCE in 2006.

Dr. Bedient has worked on a variety of hydrologic problems, including river basin analyses, major floodplain studies, groundwater contamination models, and hydrologic/GIS models in water resources. He has been actively involved in developing computer systems for flood prediction and warning, and recently directed the development of a real-time flood alert system (FAS2 and FAS3) for the Texas Medical Center (TMC) in Houston. He has built real-time flood alert systems for several communities across Texas. He currently directs the Severe Storm Prediction Education and Evacuation from Disasters (SSPEED) Center at Rice University, an eight-university research organization with private and public entities that predicts and assesses the impacts of severe storms on the Gulf Coast. This center is devoted to developing real-time flood alert and surge alert systems for the coastal areas around Houston such as the Houston Ship Channel, and also evaluates structural and non-structural methods for mitigation of severe storms.

Dr. Bedient has overseen the monitoring, modeling, and remediation at numerous hazardous waste sites, and has extensive experience in contaminant transport at sites impacted with chlorinated solvents and fuels. He has served on two National Academy of Science committees relating to environmental remediation and technology, and has received research funding from NSF, the U.S. EPA, the U.S. Department of Defense, the State of Texas, the U.S. Army Corps of Engineers, the City of Houston, and the Houston Endowment.

Wayne C. Huber is Emeritus of Civil and, Construction, and Environmental Engineering at Oregon State University, Corvallis and Senior Consultant with Geosyntec Consultants, Portland, Oregon. His doctoral work at the Massachusetts Institute of Technology dealt with thermal stratification in reservoirs, for which he received the Lorenz G. Straub Award from the University of Minnesota and the Hilgard Hydraulic Prize from the American Society of Civil Engineers (ASCE). He is a member of several technical societies and has served several administrative functions within the ASCE. He is the author of over 120 reports and technical papers, is a registered professional engineer, and has served as a consultant on numerous studies done by public agencies and private engineering firms.

Beginning at the University of Florida and continuing at Oregon State University, Dr. Huber's research has included studies of urban hydrology, stormwater management, nonpoint source runoff, river basin hydrology, lake eutrophication, rainfall statistics, and hydrologic and water quality modeling. He is one of the original authors of the EPA Storm Water Management Model and helped to maintain and improve the model following its 1971 introduction. Dr. Huber is an internationally recognized authority on runoff quantity and quality processes in urban areas.

Baxter E. Vieux Baxter E. Vieux is a Joseph A. Brandt Professor in the School of Civil Engineering and Environmental Science, University of Oklahoma, Norman. He teaches courses in hydrology, Environmental Modeling, GIS applications, water quality management, and engineering graphics and design. Before joining OU in 1990, he held a professorship at Michigan State University teaching watershed management after earning his PhD there. Prior to his academic career, he spent ten years with the USDA Natural Resources Conservation Service (formerly SCS) serving as Acting State Engineer, and was responsible for statewide engineering design and construction programs in Michigan. He is a registered professional engineer in several states and is co-principal and founder of Vieux & Associates, Inc., an engineering technology company with clients in the US and internationally in radar rainfall, GIS, and hydrology. Dr. Vieux is the innovator and architect of the first commercially available physics-based distributed hydrologic model, Vflo™, which uses real-time radar inputs for hydrologic analysis and prediction. This model is designed for both urban and rural hydrology, and is the subject of a US Patent, 7,136,756 B1, A Method for Determining Runoff. Externally sponsored academic research has been funded by NSF, USGS, NASA, EPA, NWS, NOAA, Army Corps of Engineers, NSF, and state/local agencies. He has authored over 110 publications in hydrology including a recent book in its second edition, *Distributed Hydrologic Modeling Using GIS*, Kluwer Academic Press, Vol. 48. As a hydrologist, Dr. Vieux has developed radar rainfall monitoring and runoff modeling technology, deployed operational hydrologic forecasting services, and consulted in surface water modeling for a variety of projects and clients in the US. Internationally, he has conducted research and worked on projects dealing with distributed hydrology for a number of agencies.

Chapter 1

Hydrologic Principles



Upper Yosemite Fall, Yosemite National Park.
(Photo courtesy National Park Service, U.S.
Dept. Interior.)

Hydrology is a multidisciplinary subject that deals with the occurrence, circulation, storage, and distribution of surface and ground water on the earth. The domain of hydrology includes the physical, chemical, and biological reactions of water in natural and man-made environments. Because of the complex nature of the hydrologic cycle and its relation to weather inputs and climatic patterns, soil types, topography, geomorphology, and other related factors, the boundary between hydrology and other earth sciences (i.e., meteorology, geology, oceanography, and ecology) is not distinct.

The study of hydrology also includes topics from traditional fluid mechanics, hydrodynamics, and water resources engineering (Maidment, 1993; Mays, 2001). In addition, many modern hydrologic problems include considerations of water quality and contaminant transport. Water quality topics, though important, are not included in this text due to space limitations; they have been covered in a number of modern sources on surface water quality (Huber, 1993; Chapra, 1997; Martin and McCutcheon, 1999) and ground water hydrology and contamination (Bedient et al., 1999; Fetter, 1999; Charbeneau, 2000).

1.1 INTRODUCTION TO HYDROLOGY

The **hydrologic cycle** is a continuous process in which water is evaporated from water surfaces and the oceans, moves inland as moist air masses, and produces precipitation if the correct vertical lifting conditions exist. The precipitation that falls from clouds onto the land surface of the earth is dispersed to the hydrologic cycle via several pathways (Fig. 1-1). A portion of the **precipitation** P , or rainfall, is retained on the soil near where it falls and returns to the atmosphere via **evaporation** E , the conversion of water to water vapor from a water surface, and **transpiration** T , the loss of water vapor through plant tissue and leaves. The combined loss, called **evapotranspiration** ET , is a maximum value if the water supply in the soil is adequate at all times. These parameters are further discussed in subsequent sections of this chapter and Section 2.6.

Some water enters the soil system as **infiltration** F , which is a function of soil moisture conditions and soil type, and may reenter channels later as interflow or may percolate to recharge the shallow ground water. **Ground water** G flows in porous media in the subsurface in either shallow or deeper aquifer systems that can be pumped for water supply to agricultural and municipal water systems (see Chapter 8).

The remaining portion of precipitation becomes overland flow or **direct runoff** R , which flows generally in a down-gradient direction to accumulate in local streams that then flow to rivers. Hydrologic analysis to determine runoff response from a watershed area is covered in Chapter 2. Evaporation and infiltration are both complex losses from input rainfall and are difficult to measure or compute from theoretical methods, covered in detail in Sections 2.6, 2.7, and 2.8.

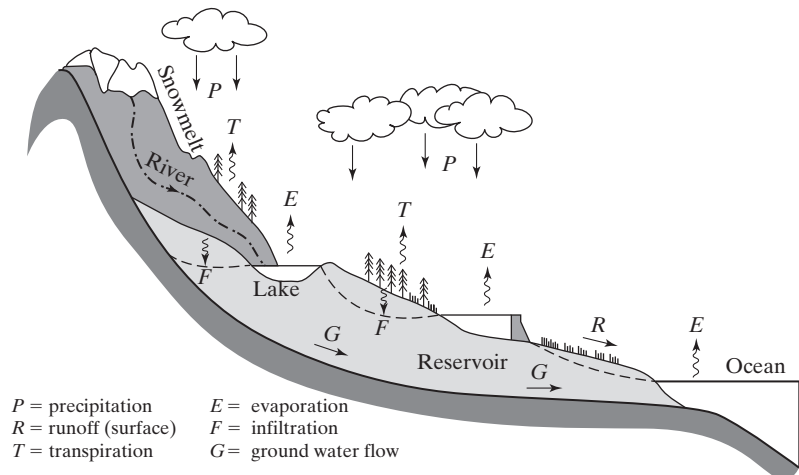
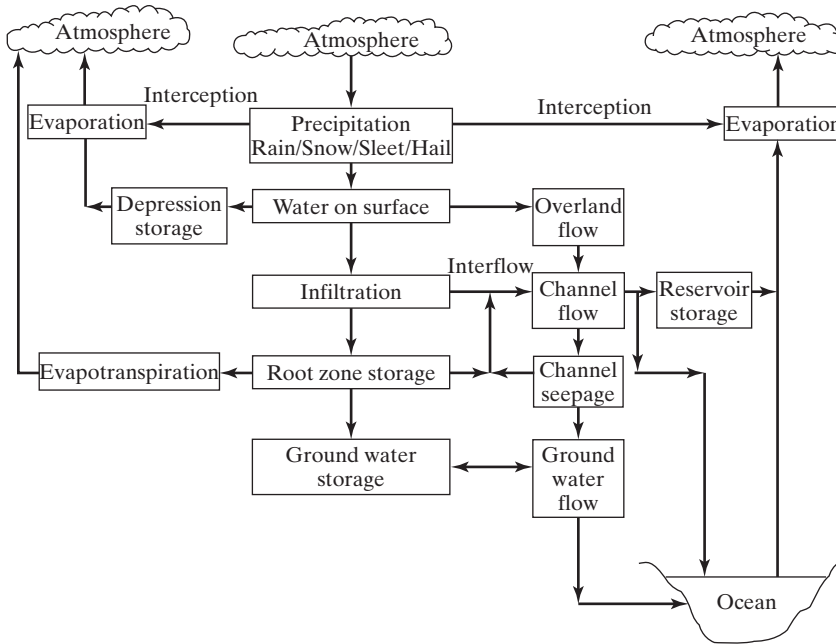


Figure 1-1a

The hydrologic cycle discharges surface water and groundwater from the higher elevation to the lower elevation.

**Figure 1-1b**

Flow chart of the components of the hydrologic cycle.

Surface and ground waters flow from higher elevations toward lower elevations and may eventually discharge into the ocean, especially after large rainfall events (Fig. 1-1a). However, large quantities of surface water and portions of ground water return to the atmosphere by evaporation or *ET*, thus completing the natural hydrologic cycle (Fig. 1-1b). Precipitation from the atmosphere is a major force that drives the hydrologic cycle, and understanding major weather parameters and systems is important for the prediction of precipitation events (see Section 1.3).

Ancient History

Biswas (1972), in a concise treatment of the history of hydrology, describes the early water management practices of the Sumerians and Egyptians in the Middle East and the Chinese along the banks of the Huang He (Yellow River). Archeological evidence exists for hydraulic structures that were built for irrigation and other water control activities. A dam was built across the Nile about 4000 B.C., and later a canal for fresh water was constructed between Cairo and Suez.

The Greek philosophers were the first serious students of hydrology, with Aristotle proposing the conversion of moist air into water deep inside mountains as the source of springs and streams. Homer suggested the idea



Figure 1–2

Roman aqueducts, known as Pond du Gard located in Southern France, crosses the River Gardon (Gard).

of an underground sea as the source of all surface waters. The Romans constructed numerous aqueducts to serve large cities as well as small towns and industrial sites. The Romans had the largest collection with water being supplied by 11 aqueducts constructed over a period of about 500 years. Figure 1–2 shows one of the famous aqueducts built in France during that early period. They served potable water and supplied the numerous baths and fountains in the city, as well as finally being emptied into the sewers, where the once-used gray water performed their last function in removing wastes. The construction of the Roman aqueducts is considered one of the most important engineering feats in history.

Streamflow measurement techniques were first attempted in the water systems of Rome (A.D. 97) based on the cross-sectional area of flow. It remained for Leonardo da Vinci to discover the proper relationship between area, velocity, and flow rate during the Italian Renaissance. The first recorded measurement of rainfall and surface flow was made in the seventeenth century by Perrault. He compared measured rainfall to the estimated flow of the Seine River to show the two were related. Perrault's findings were published in 1694. Halley, the English astronomer (1656–1742), used a small pan to estimate evaporation from the Mediterranean Sea and concluded that it was enough to account for tributary flows. Mariotte gaged the velocity of flow in the Seine River in Paris. These early beginnings of the science of hydrology provided the foundation for numerous advances in the eighteenth century, including Bernoulli's theorem, the Pitot tube for measuring velocity, and the Chezy (1769) formula, which form the basis for modern hydraulics and fluid measurement.

During the nineteenth century, significant advances in ground water hydrology and hydraulics occurred. Darcy's law for flow in porous media was a major advance, as well as the Dupuit–Thiem well flow formula (Chapter 8). In addition, the Hagen–Poiseuille capillary flow equation was developed to describe flow in small channels. The Darcy–Weisbach equation for pipe flow was also developed during this same period in the 1850s. In surface water hydrology, many flow formulas and measuring instruments

were developed that allowed for the beginning of systematic stream gaging. In 1867, discharge measurements were organized on the Rhine River at Basel and quickly expanded throughout Europe.

The U.S. Geological Survey set up the first systematic program of flow measurement in the United States on the Mississippi in 1888. During this same period, the United States founded a number of hydrologic agencies, including the U.S. Army Corps of Engineers (1802), the U.S. Geological Survey (USGS, 1879), the Weather Bureau (1891), and the Mississippi River Commission (1893). The Price current meter was invented in 1885, and Manning's formula was introduced in 1889 (Manning, 1889). The Weather Bureau is now called the National Weather Service (NWS) and is one of six organizations underneath the National Oceanic and Atmospheric Administration (NOAA). NOAA is the agency responsible for weather data collection and severe-storm, river, and hurricane forecasting for the United States, and many of its websites are listed throughout the textbook. The USGS gaging network for rainfall, streamflow, and water quality is one of the most extensive in the world.

Early History (1930s–1950s)

The period from 1930 to 1950, which Chow (1964) called the Period of Rationalization, produced a significant step forward for the field of hydrology, as government agencies began to develop their own programs of hydrologic research. Sherman's unit hydrograph (1932) (see Chapter 2), Horton's infiltration theory (1933), and Theis's nonequilibrium equation (1935) in well hydraulics (Chapter 8) advanced the state of hydrology in very significant ways. Gumbel (1958) proposed the use of extreme-value distributions for frequency analysis of hydrologic data, thus forming the basis for modern statistical hydrology (Chapter 3). In this period, the U.S. Army Corps of Engineers (ACOE), the NWS within NOAA, the U.S. Department of Agriculture (USDA), and the USGS made significant contributions in hydrologic theory and the development of a national network of gages for precipitation, evaporation, and streamflow measurements. The NWS is still largely responsible for rainfall measurements, reporting and forecasting of severe storms, and other related hydrologic investigations.

The U.S. ACOE and the USDA Soil Conservation Service (now called the Natural Resources Conservation Service [NRCS]) made significant contributions to the field of hydrology in relation to flood control, reservoir development, irrigation, and soil conservation during this period. More recently, the USGS has taken significant strides to set up a national network of stream gages and rainfall gages for both quantity and quality data. Their water supply publications and special investigations have done much to advance the field of hydrology by presenting the analysis of complex hydrologic data to develop relationships and explain hydrologic processes. The NWS and USGS both support numerous websites for the dissemination of watershed information and precipitation and streamflow data from thousands

of gages around the country. Many of these sites are listed in Appendix E and on the textbook website (<http://hydrology.rice.edu/bedient>).

The government agencies in the United States have long performed vital research themselves, providing funding for private and university research in the hydrologic area. Many of the water resources studies and large dam, reservoir, and flood control projects in the 1930s and 1940s were a direct result of advances in the fields of fluid mechanics, hydrologic systems, statistical hydrology, evaporation analysis, flood routing, and operations research. Many of the advances from that era continue to this day as the methods to predict runoff, infiltration, and evaporation have not changed much in over 50 years. Major contributions from Horton (1933, 1940, 1941) and from Penman (1948) in understanding hydrologic losses were related to the water and irrigation needs of the agricultural sector in the United States following the devastation of the dust bowl era of the 1930s.

Major water resources projects built during the 1930s were a direct result of major floods on the Mississippi River and the economic depression across the nation. The building of the massive Hoover Dam on the Colorado River for flood and sediment control and water supply in the early 1930s provided employment for over 40,000 and was the largest construction project ever conceived to that point (see Chapters 12 and 13).

Modern History

In the 1950s and 1960s, the tremendous increase of urbanization following World War II in the United States and Europe led to better methods for predicting peak flows from floods, for understanding impacts from urban expansion, and for addressing variations in storage in water supply reservoirs. Major expansion of cities and water systems within the United States during the 1950s led to a need for better understanding of floods and droughts, especially in urban areas. Water resource studies became an everyday occurrence in many rapidly developing areas of the United States, tied to the expansion of population centers in the southern, southwestern, and western states. Hydrologic analyses presented in detail in Chapters 2 through 9 in the text were a major component of many of these studies.

During the 1970s and early 1980s, the evaluation and delineation of floodplain boundaries became a major function of hydrologists, as required by the Federal Emergency Management Agency (FEMA) and local flood control or drainage districts. In order for communities to be eligible for flood insurance administered by FEMA, they are required to delineate floodplain boundaries using hydrologic analysis and models. Floodplain analysis is covered in detail in Chapters 5, 7, 9, and 12. This function has taken on a vital role in many urban areas, as damages from severe floods and hurricanes continue to plague the United States, especially in coastal and low-lying areas. The period from 2004 to 2010 accounted for numerous hurricanes that caused massive damages and deaths in several areas, especially along the

coastlines of Texas, Louisiana, Mississippi, Alabama, and Florida. The massive Mississippi flood of 1993 wreaked havoc within the central United States, and was repeated in 2011 with major devastation to states from Illinois south to Louisiana (Chapter 12).

In recent years, the traditional approaches to flood control have been reassessed. A study titled “Higher Ground” from the National Wildlife Federation (1998) found a number of communities with large numbers of repetitive flood losses, such as New Orleans and Houston. Since the great midwestern flood of 1993, there has been a significant shift in national flood policy away from using only structural solutions, such as levee and channel construction. Flood damage from Tropical Storm Allison in Houston in 2001 was a major wake-up call for better protection and warning systems in critical urban areas. The massive devastation from Hurricane Katrina in New Orleans in August 2005 and Hurricane Ike in 2008 in Houston-Galveston will provide long-lasting incentive to improve our ability to warn for and recover from such severe storms. Modern methods for structural flood control, as well as nonstructural approaches, better management of flood-prone areas, and voluntary property buyouts, must be considered in any overall flood management plan (Chapter 12). Chapter 13 explores several major water resources projects across the United States and Asia in terms of engineering significance as well as associated environmental and policy impacts on the nation.

Computer Advances

The introduction of the digital computer into hydrology during the 1960s and 1970s allowed complex water problems to be simulated as complete systems for the first time. Large computer models can now be used to match historical data and help answer difficult hydrologic questions (Singh and Frevert, 2006). The development of these tools over the past few decades has helped direct the collection of the hydrologic data to calibrate, or “match,” the models against observation. In the process, the understanding of the hydrologic system has been greatly advanced. Hydrologic computer models developed in the 1970s have been applied to areas previously unstudied or only empirically defined. For example, urban stormwater, floodplain and watershed hydrology, drainage design, reservoir design and operation, flood frequency analysis, and large-river basin management have all benefited from the application of computer models.

Hydrologic simulation models applied to watershed analysis are described in detail in Chapter 5. Single-event models such as HEC-HMS are used to simulate or calculate the resulting storm hydrograph (discharge vs. time) from a well-defined watershed area for a given pattern of rainfall intensity. Continuous models such as the Hydrological Simulation Program—Fortran (HSPF) and the Storm Water Management Model (SWMM) can account for soil moisture storage, evapotranspiration, and antecedent

rainfall over long time periods. Statistical models can be used to generate a time series of rainfall or streamflow data, which can then be analyzed with flood frequency methods.

Newer distributed hydrologic models (i.e., VFLO and the MIKE series of models) can handle input, output, and data manipulation at the watershed level (see Chapters 5, 10, 11, and 12). Unquestionably new digital approaches combined with distributed terrain modeling have revolutionized hydrology in recent years, just as the original wave of models did in the decade of the 1970s. Also faster computers and available datasets have been instrumental in advancing the field.

The data revolution in hydrology and geographical information systems (GIS) have made available newer and more accurate datasets on topography, slope, rainfall, soils, land use, and channel characteristics for many areas. Moreover, most hydrological and meteorological data may be retrieved online from agencies such as the USGS and NWS, and various county and municipal sources. These datasets, combined with existing simulation models in hydrology, if applied correctly, provide the most accurate approach to understanding complex water resources systems, and a new era in the science of hydrology has begun this decade. New design and operating policies are being advanced and implemented that could not have been realized or tested before without the aid of sophisticated computer models linked with digital data.

1.2 WEATHER SYSTEMS

The atmosphere is the major hydrologic link between oceans and continents on the planet, facilitating the cycle of water movement on earth. The hydrologic cycle is shaped by the conditions of the atmosphere, with precipitation as the main input to the cycle. Water vapor content is both a major catalyst and a balancing factor of atmospheric processes that create the weather in the lower atmosphere. The following section reviews major elements of atmospheric processes that directly impact the hydrologic cycle. More details on atmospheric processes can be found in modern meteorology and hydrology textbooks (Anthes, 1997; Ahrens, 2000; Dingman, 2002).

Atmospheric Parameters

Pressure is defined as the force per unit area exerted on a surface, and **atmospheric pressure** measures the weight of the air per unit area. Average air pressure at sea level is approximately 1 atmosphere, or 1013 millibars (mb) or 14.7 lb/in.² or 760 mm-Hg or 29.97 in.-Hg. Note that 1 mb = 10² pascals (Pa), where 1 Pa = 1 N/m². As elevation increases and the density of air molecules decreases, atmospheric pressure also decreases. The horizontal and vertical pressure variations that occur due to low- and high-pressure systems are responsible for wind, and help drive much of our weather. Absolute temperature (T), pressure (P), and density (ρ) are related through the

ideal gas law, $P = \rho RT$, where R is the gas constant for the gas in question (see Eq. 1–1). At constant density, temperature is directly proportional to pressure; thus, with an increase in temperature comes an increase in pressure. Air pressure is proportional to density, so that in the atmosphere a decrease in temperature causes an increase in the density of the air molecules. Cold air masses are generally associated with the higher atmospheric pressure.

Humidity is a measure of the amount of water vapor in the atmosphere and can be expressed in several ways. Specific humidity is the mass of water vapor in a unit mass of moist air. The **relative humidity** is a ratio of the air's actual water vapor content compared to the amount of water vapor at saturation for that temperature. The partial pressure of water vapor is the contribution made by water to the total atmospheric pressure. When a volume reaches its maximum capacity for water vapor, the volume is said to be saturated and can accept no more vapor. This vapor pressure is known as **saturation vapor pressure**. Vapor pressure is dependent on temperature; and as air is lifted and cools, its relative humidity increases until saturation, and then condensation of water vapor to liquid water can occur. The temperature to which a sample of air must be cooled to reach saturation is defined as the **dew point temperature**. These concepts are described in more detail later in this section.

Water vapor has the ability, unique among gases, to change from one state of matter to another (solid, liquid, or gas) at the temperatures and pressures that typically exist on earth. A change in phase (e.g., from liquid to vapor) requires that heat be released or absorbed. The processes of converting solid ice to liquid water, called melting, and water to vapor, called evaporation, both require significant heat exchange. It takes approximately 600 cal to convert 1 g of water to water vapor. When such changes take place, the heat is absorbed and no temperature change takes place. The heat used in this process is latent heat. Condensation is the process in which water vapor changes into a liquid state. For this to occur, energy must be released in an amount equivalent to what was absorbed during evaporation. This latent heat often becomes the source of energy for severe thunderstorms, tornadoes, and hurricanes.

The Atmosphere and Clouds

Atmospheric weather systems are fueled by solar input and characterized by air masses in motion, circulating winds, cloud generation, and changes in temperature and pressure. Lifting mechanisms are required for moist air masses to cool and approach saturation conditions. As a result of the interaction of rising air masses with atmospheric moisture, the presence of small atmospheric nuclei, and droplet growth, precipitation in the form of rain, snow, or hail can result. The exact mechanisms that lead to precipitation are sometimes quite complex and difficult to predict for specific areas. But precipitation remains

as the main input to the hydrologic cycle, and the hydrologist needs a general understanding of the mechanisms that cause its formation.

Horizontal variations in atmospheric pressure cause air to move from higher pressure toward lower pressure, resulting in the generation of wind. Vertical displacement causes air to move as well, but at a far slower rate than horizontal winds. The vertical movement and lifting of air results in the formation of clouds. Clouds are familiar to all of us, and represent collections of small droplets of water or tiny crystals of ice. The names of the basic clouds have the following roots:

cirrus, feathery or fibrous clouds

stratus, layered clouds

cumulus, towering, puffy clouds

alto, middle-level clouds

nimbus, rain clouds

The second aspect of cloud classification is by height. Anthes (1997) presents a detailed coverage of cloud types for the interested student. One type of high cloud of importance in hydrology is the **cumulonimbus**, one often found in heavy thunderstorms that produce massive rainfall. Cirrus clouds are very high collections of ice crystals and often indicate the approach of a cold front and that weather is about to change. While clouds result when air rises and cools, surface fog results from cooling near the surface or from the addition of enough water vapor to cause saturation. Fog is essentially a low cloud with a base that is very near the ground, often reducing the visibility in the area within or around it. Marine fog is common along the California and upper Atlantic coasts in the United States.

General Circulation

The general circulation of wind across the earth is caused by the uneven heating of earth's surface through solar input, and by the earth's rotation. At the equator, solar radiation input and temperature are greatest because of the shape and tilt of the globe relative to the sun. Three latitudinal circulation cells transport heat from the equator to the poles (Fig. 1–3). As warm air travels northward in the middle latitudinal cell on a spinning earth, it tends to shift to the right (toward the east) in the northern hemisphere due to the Coriolis force, thus causing the occurrence of winds called **westerlies**. These winds tend to drive the direction of major weather systems from west to east across major portions of the continental United States.

Between 30 degrees north latitude and the equator, the flow is generally toward the south and is altered to create the trade winds (**easterlies**) by the Coriolis force in the northern hemisphere. The trade winds allowed explorers from Europe to sail across the ocean to the New World. The Coriolis force causes the flow along latitude circles (east/west) to be 10 times greater than

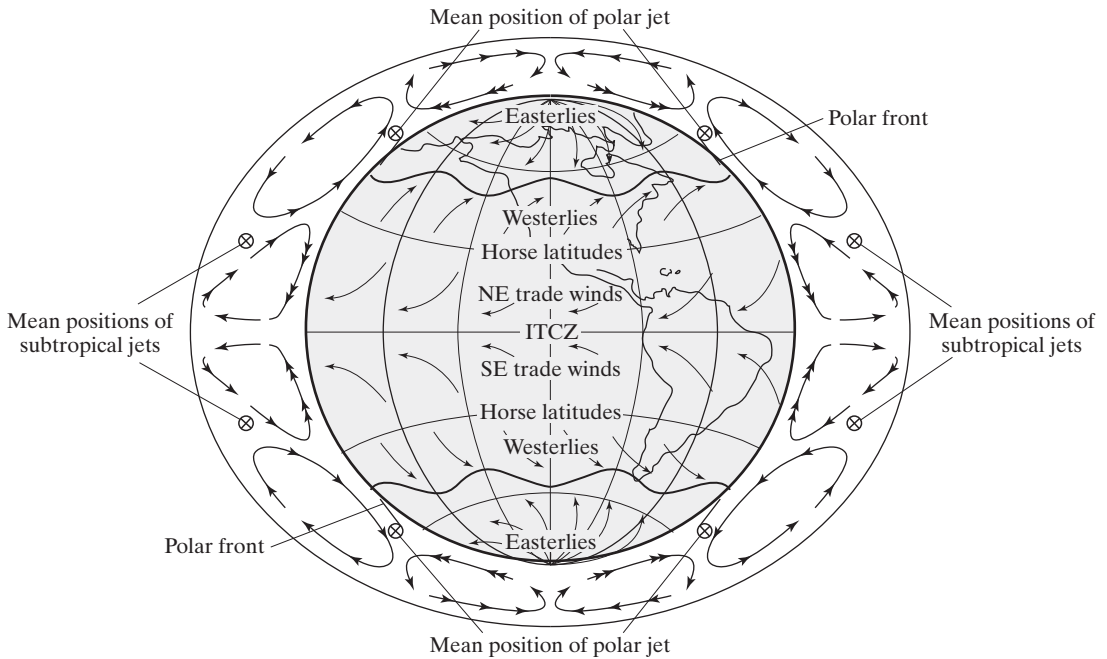


Figure 1-3
General circulation of currents and wind patterns across the earth.

the flow along the meridians (north/south). Around the 30 degrees north and south latitudes, descending air creates a region of minimal winds and little cloudiness that is known as the horse latitudes. Near the equator is another region of light and variable winds called the doldrums, or the intertropical convergence zone. This is the area of maximum solar heating, where surface air rises and flows toward both poles.

Jet streams, first observed in 1946, are narrow bands of high-speed winds that circle each hemisphere like great rivers, at elevations extending from 2.5 or 3 miles to above the tropopause. The polar and subtropical jets are associated with the polar front near 45 degrees latitude and 30 degrees latitude, respectively. The jet streams are highly variable and can flow at speeds as high as 100 mph faster than the air on either side of them. Jet streams have a major impact as the driving force for weather systems across the United States, especially in the winter season.

Air Masses and Fronts

Air masses are large bodies of air with fairly consistent temperature and humidity gradients in the horizontal direction at a given altitude. Air masses dominate our weather and are classified in two ways: the source from which

they were generated, land (continental) or water (maritime), and the latitude of generation (polar or tropical). The four combinations are designated cP, cT, mP, and mT.

Each of these types of air masses is present in the United States. The continental polar air mass emanates from Canada and passes over the northern United States, often dropping significant amounts of rain and snow on areas downwind of the Great Lakes. The maritime polar air mass also comes southward from the Atlantic coast of Canada and affects the New England states. Another maritime polar air mass comes from the Pacific and hits the extreme northwestern states. The maritime tropical air masses come from the Pacific, the Gulf of Mexico, and the Atlantic. The entire southern United States is affected by these air masses. The only time continental tropical air masses form is during the summer, and they originate in Texas and affect the states bordering to the north.

The boundary between one air mass and another is called a frontal zone, or **front**. When two air masses meet, the front will slope diagonally, as the colder, denser air mass pushes under the warmer air mass. Between the two fronts, a transition zone occurs, usually 30 to 60 miles wide. Whether the masses are traveling against each other or in the same direction, the warmer air mass will be forced upward and cooled by expansion. Since cooling a parcel of air lowers its saturation vapor pressure, this may cause condensation and the development of precipitation often associated with frontal systems (Fig. 1-4). The NWS website provides an amazing amount of weather information and satellite data all on one place (<http://www.nws.noaa.gov/>).

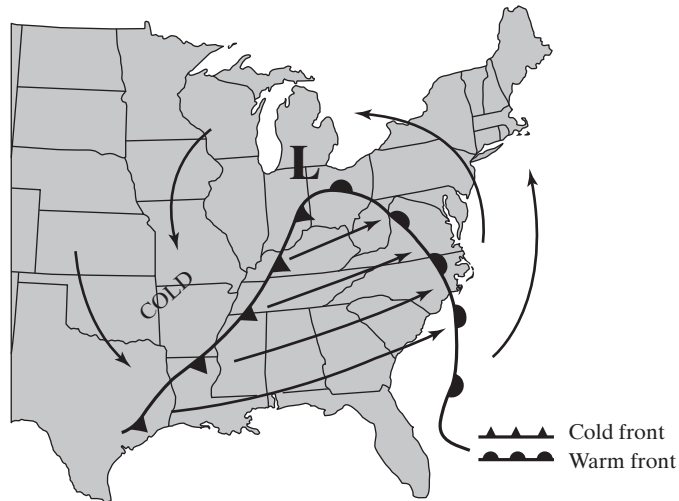


Figure 1-4

Direction of the cold and warm fronts in the eastern United States. The cold fronts come from the North toward the Gulf and the warm fronts start from the Gulf to push inland.

Development of surface cyclones along fronts occurs when an upper-level disturbance approaches a front. The upper-level patterns of convergence and divergence produce pressure changes at the surface, which then produce low-level circulation (**wave cyclone**). As a wave cyclone develops, low pressure forms at its apex and both the warm and cold currents move in a cyclonic pattern (counterclockwise in the northern hemisphere) around it. To the left of the apex, the cold front is advancing towards the warm air, and the warm front is receding to the right. The warm air between the fronts is known as the warm sector. The entire system generally moves toward the right (eastward), and ahead of the warm front the first sign of the approaching system is high cirrus clouds. As the center of low pressure approaches, the pressure falls and the wind increases and changes its direction to counterclockwise. The temperature begins to decrease as the frontal zone approaches. Within several hundred miles of the surface position of the front, precipitation begins, either rain or snow. Fronts can be fast moving in the winter or can be slowed or stalled due to the presence of other air masses or high-pressure systems in the fall or spring (Anthes, 1997). Warm fronts can also generate rainfalls as they move across an area.

Fronts are a major factor in U.S. weather patterns, especially from September through April in most years. The type of weather accompanying the passage of the cold front depends on the front's sharpness, its speed, and the stability of the air being forced aloft. Often there are towering cumulus clouds and showers along the forward edge of the front. Sometimes, especially in the Midwest during the spring, several **squalls**, or a strong line of storms, precede the front. Tornadoes can form in these storm cells, especially in areas of north Texas, Oklahoma, and Arkansas. But in other cases, nimbostratus clouds and rain extend over a zone of 50 to 60 miles. After the frontal passage, the wind changes sharply and the pressure begins to rise. Within a short distance behind the cold front, the weather clears, the temperature begins to fall, and the visibility greatly improves.

Thunderstorms

Thunderstorm activity is characterized by cumulonimbus clouds that can produce heavy rainfall, thunder, lightning, and occasionally hail. Thunderstorms are the result of strong vertical movements in the atmosphere and usually occur in the spring or summer in the United States. They require warm, moist air, which when lifted will release enough latent heat to provide the buoyancy needed to maintain its upward motion. Accordingly, they generally occur in warm air masses that have become unstable either through extreme low-pressure systems, surface heating, or forced ascent over mountains. The geographic pattern of thunderstorm occurrence in the United States is a result of both an area's distance from source air masses and its topography. Florida and the Gulf Coast are affected most frequently, sometimes as often as 100 times in a year.



Figure 1–5

The Cumulonimbus cloud that signals that a storm is approaching.

Thunderstorms develop in three characteristic stages. The first is the cumulus stage, when moist air rises and cools and condenses into a cumulus cloud. The cumulus cloud then continues to grow taller as the rising air condenses at successively higher levels (Fig. 1–5). The diameter of the storm cell grows in width from about 1 mile to 6 to 9 miles and vertically to 5 or 6 miles. The rising air is no longer able to retain water droplets and rain begins to fall.

The rain marks the beginning of the second stage of the thunderstorm, the mature stage. During the mature stage, the large water particles or hail in the clouds begin to fall because they have become too large to be supported by the updraft. As this happens, drier air around the cloud is being drawn into it, in a process known as entrainment. This drying of the air results in the evaporation of some rain drops, which cools the air. The air is now colder and heavier than the air around it and, while the upper part of the cloud still has a strong updraft, a lower part of the storm cloud begins to descend as a downdraft. This downdraft eventually reaches the ground and spreads away from the thunderstorm, causing the cool gusts of wind that usually foreshadow the arrival of a thunderstorm. Meanwhile, the upper part of the cloud reaches a stable part of the atmosphere, and high-altitude winds may create a typical anvil shape (Fig. 1–6). The cloud reaches its greatest vertical development in this stage, extending upward over 7.5 mi (40,000 ft, 12 km). Lightning, turbulence, heavy rains, and, if present, hail are all found at this time. The second stage is the most intense period of the thunderstorm.

When the downdraft has spread over the entire storm cell and the updraft has been cut off, the storm begins its final stage, the dissipating stage. The rate of precipitation diminishes and so the downdrafts are also gradually subdued. The final flashes of lightning fade away and the cloud begins to

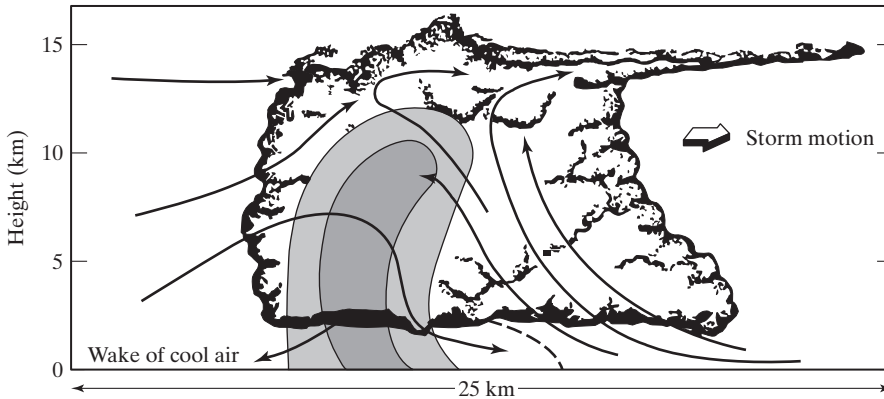


Figure 1-6

Typical thunderstorm cloud evolution. The typical anvil-shaped clouds that are present during a thunderstorm are caused by the movement of cold air and warm air. As the cold air moves downward and the warm air moves upward, the warm air above spreads out in order to cool, resulting in the following shape, very much like an anvil.

dissolve or perhaps persist a while longer in a stratified form (see Fig. 1-6). Intense thunderstorms are of great interest, since they can produce significant amounts of rainfall in a short time period. Chapter 11 discusses new radar methods for the detection of severe storms and for the measurement of rainfall intensities associated with severe storms.

Hurricanes

Tropical cyclones or **hurricanes** are intense cyclonic storms that form over the tropical oceans, between 5 and 20 degrees latitude. With extreme amounts of rainfall and winds that can exceed 186 mph (300 km/hr), tropical cyclones are the most destructive storms on earth. The local name for this storm varies throughout the world: typhoon in Eastern Asia, cyclone in India, and baguio in the China Sea. The North American term, used in this discussion, is hurricane. By international agreement, a storm is a hurricane if it has wind speeds of at least 74 mph (119 km/hr) and a rotary circulation. When its wind speeds are between 39 mph (63 km/hr) and 73 mph (119 km/hr), it is a tropical storm. Tropical disturbances with winds that do not exceed 39 mph (61 km/hr) are tropical depressions. All tropical storms and hurricanes are given proper names in alphabetical order, starting at the beginning of the alphabet when the storm season begins on June 1 and starting over during the next season. Hurricanes are classified according to a scale based on central pressure, storm surge height, and wind speed. The scale has five categories, ranging from category 1, a hurricane of minimal damage, to category 5, a hurricane of catastrophic proportions (see Table 1-1).

Table 1–1. The Saffir-Simpson Hurricane Wind Scale

Category	Wind Speed (mph)	Extent of Damage	Damage Description
Tropical Storm	39–73	Minor	Some flooding
1	74–95	Minimal	Damage limited to unanchored mobile homes, shrubbery, and trees
2	96–110	Moderate	Some roof, door and window damage to buildings, some trees blown down
3	111–130	Extensive	Some structural damage to residences and utility buildings, trees defoliated and many blown down
4	131–155	Extreme	Extensive curtainwall failures and some complete roof failures, shrubs, trees, and all signs blown down
5	156+	Catastrophic	Complete roof failure and some complete building failures

The warm, moisture-laden air of the tropical oceans possesses an enormous capacity for heat energy, and most of the energy required to create and sustain a hurricane comes from what is released through condensation from vapor to water. Hurricanes develop most often during the late summer when the oceans are warm (26°C or higher) and are thus able to provide the necessary heat and moisture to the air. The hurricane season in the West Indies extends from June 1 to November 1, but 84% of the hurricanes and tropical cyclones below hurricane intensity reported from 1887 to 1958 in the North Atlantic occurred during August, September, and October. There is considerable variability in the number of hurricanes in the Atlantic annually. In the 40-year period from 1950 to 1990, the number of hurricanes in the Atlantic varied from 3 to 12 per year.

The decades of the 1990s and 2000s have seen especially high hurricane activity in the North Atlantic basin, including both hurricane frequency and intensity. As indicated below, 2004 and 2005 were very active hurricane years for the coastal United States. Figure 1–7 shows the devastating track of Hurricane Ike that damaged the Houston-Galveston coastline in September, 2008. Statistics have shown that the number of tropical storms correlates with several climatological anomalies on a global scale, including rainfall in West Africa in the prior year, the direction of the winds in the stratosphere, and the **El Niño-Southern Oscillation (ENSO)** phenomenon, characterized by a warm phase associated with high sea surface temperatures off the coast of Peru, low atmospheric pressure over the eastern Pacific, high pressure in the western Pacific, and strong winds aloft over the tropical Atlantic (creating high vertical wind shear and unfavorable conditions for hurricane development). A cold phase (**La Niña**) has low sea surface temperatures in the

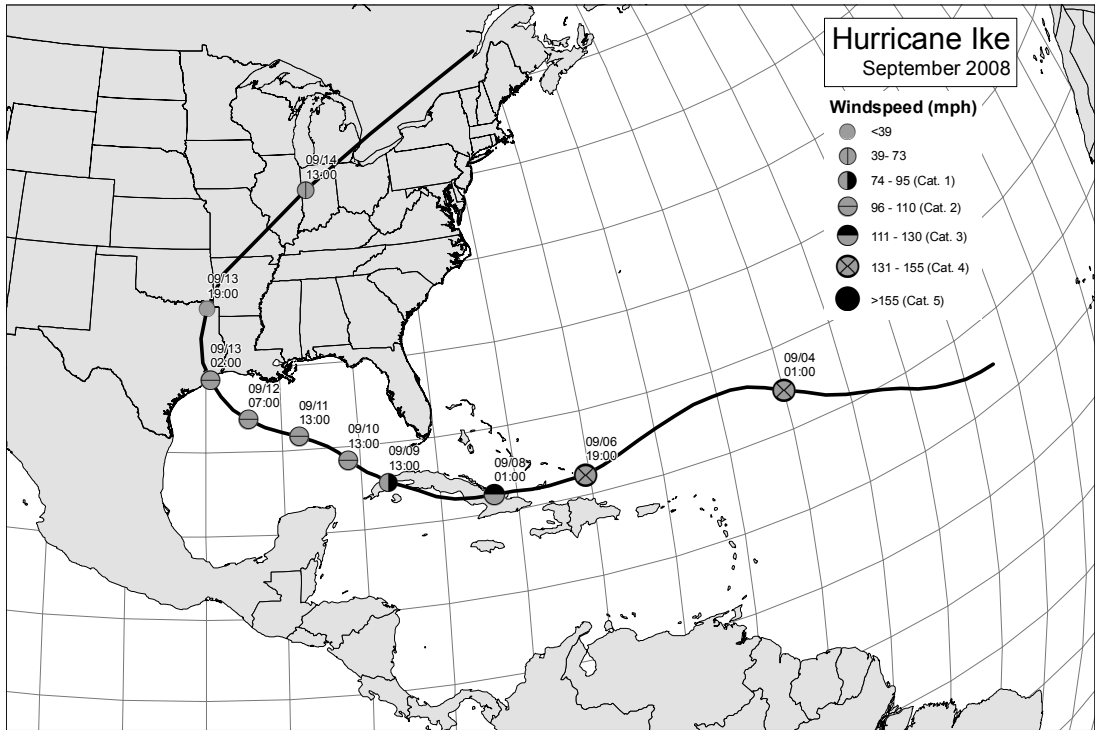


Figure 1-7

Hurricane Ike's track. The hurricane started in the Atlantic and made its way into the Gulf. What started out as a Category 4 storm in the Atlantic became a Category 3 as it hit Cuba and finally weakened to a Category 2 storm as it made landfall in Texas.

eastern Pacific and the opposite pressure and wind anomalies. Meteorology textbooks and websites usually provide more details on the occurrence and characteristics of hurricanes (Anthes, 1997; Ahrens, 2000; Dingman, 2002). More details can be found in Chapter 12.

Moisture Relationships

Atmospheric moisture is a necessary source for precipitation and is generally provided from evaporation and transpiration. Precipitation across the United States is largely due to proximity of evaporation from oceans and the Gulf of Mexico and to subsequent transport over the continent by the atmospheric circulation system. Common measures of atmospheric moisture, or humidity, include vapor pressure, specific humidity, mixing ratio, relative humidity, and dew point temperature. (Most of these terms were defined earlier in the chapter.) Under moist conditions, water vapor can be assumed to obey the ideal gas law, which allows derivation of simple relations between pressure, density, and temperature.

The partial pressure is the pressure that would be exerted on the surface of a container by a particular gas in a mixture. The partial pressure exerted by water vapor is called **vapor pressure** and can be derived from Dalton's law and the ideal gas law as

$$e = \frac{\rho_w RT}{0.622}, \quad (1-1)$$

where

$$\begin{aligned} e &= \text{vapor pressure (mb)}, \\ \rho_w &= \text{vapor density or absolute humidity (g/cm}^3\text{)}, \\ R &= \text{dry air gas constant} = 2.87 \times 10^3 \text{ mb cm}^3/\text{g}^\circ\text{K}, \\ T &= \text{absolute temperature (}^\circ\text{K)}. \end{aligned}$$

The factor 0.622 arises from the ratio of the molecular weight of water (18) to that of air (29). Near the earth's surface, the water vapor pressure is 1% to 2% of the total atmospheric pressure, where average atmospheric pressure is 1013.2 mb (1 mb = 10^2 pascals (Pa)). **Saturation vapor pressure** is the partial pressure of water vapor when the air is completely saturated (no further evaporation occurs) and is a function of temperature.

Relative humidity (RH) is approximately the ratio of water vapor pressure to that which would prevail under saturated conditions at the same temperature. It can also be stated as $RH = 100 e/e_s$. Thus, 50% relative humidity means that the atmosphere contains 50% of the maximum moisture it could hold under saturated conditions at that temperature. Typical relative humidity averages (high and low percentages) for eight major American cities are as follows: Houston (89, 67), Seattle (84, 62), Chicago (80, 64), New York City (72, 56), Miami (83, 61), Denver (67, 40), Albuquerque (58, 29), and Las Vegas (40, 21).

Specific humidity is the mass of water vapor contained in a unit mass of moist air (g/g) and is equal to ρ_w/ρ_m , where ρ_w is the vapor density and ρ_m is the density of moist air. Using Dalton's law and assuming that the atmosphere is composed of only air and water vapor, we have

$$\rho_m = \frac{(P - e) + 0.622e}{RT} = \frac{P}{RT} (1 - 0.378e/P). \quad (1-2)$$

Equation (1-2) shows that moist air is actually lighter than dry air for the same pressure and temperature. Thus,

$$q = \frac{\rho_w}{\rho_m} = \frac{0.622e}{P - 0.378e}, \quad (1-3)$$

where

$$\begin{aligned} q &= \text{specific humidity (g/g)}, \\ e &= \text{vapor pressure (mb)}, \\ P &= \text{total atmospheric pressure (mb)}, \\ \rho_m &= \text{density of mixture of dry air and moist air (g/cm}^3\text{)}. \end{aligned}$$

Finally, the **dew point temperature** T_d is the value at which an air mass just becomes saturated ($e = e_s$) when cooled at constant pressure and moisture content. An approximate relationship for saturation vapor pressure over water e_s as a function of temperature T is

$$e_s = 2.7489 \times 10^8 \exp\left(-\frac{4278.6}{T + 242.79}\right), \quad (1-4)$$

where e_s is in mb and T is in $^{\circ}\text{C}$. The relationship is accurate to within 0.5% of tabulated values (List, 1966) over a range of temperatures from 0°C to 40°C . Homework problems for Chapter 1 and the following Example 1.1 explore the use of Eq. (1-4) in more detail. More details on their application and use can be found in standard textbooks (Wallace and Hobbs, 1977; Ahrens, 2000; Dingman, 2002).

At the airport, weather specialist measured the air pressure to be 124.3 kPa, the air temperature was 28°C , and the dew point temperature was 20°C . Calculate the corresponding vapor pressure, relative humidity, and specific humidity. First compute e and e_s .

EXAMPLE 1-1

Air pressure = 124.3 kPa = 1243 mb

SOLUTION

Air temp = 28°C

$T_d = 20^{\circ}\text{C}$

*Hint: 100 Pa = 1 mb

Vapor Pressure

$e = 2.749 \times 10^8 \exp\left(\frac{-4278.6}{T_d + 242.79}\right)$, plug in 20°C for T to get 23.34 mb

$e_s = 2.7489 \times 10^8 \exp\left(-\frac{4278.6}{T + 242.79}\right)$, plug in 28°C for T to get 37.56 mb

Relative Humidity

$$H = 100 \frac{e}{e_s} = 100 \cdot \frac{23.34}{37.56} = 100 \cdot 0.62 = 62\%$$

Specific Humidity

$q = \frac{\rho_w}{\rho_m} = \frac{0.622e}{P - 0.378e}$, plug in 23.34 mb for e and 1243 mb for P to get 0.0117 kg

water/kg moist air

Atmospheric Stability and Phase Changes

In order for vapor to condense to water to begin the formation of precipitation, a quantity of heat known as latent heat must be removed from the moist air. The **latent heat** of condensation L_c is equal to the latent heat of evaporation L_e , the amount of heat required to convert water to vapor at the same temperature. With T measured in $^{\circ}\text{C}$,

$$L_e = -L_c = 597.3 - 0.57(T - 0^{\circ}\text{C}), \quad (1-5)$$

where L_e is in cal/g. The latent heat of melting and freezing are also related:

$$L_m = -L_f = 79.7,$$

where L_m is also in cal/g. Thus, it takes about 7.5 times the energy to evaporate a gram of water compared to melting a gram of ice.

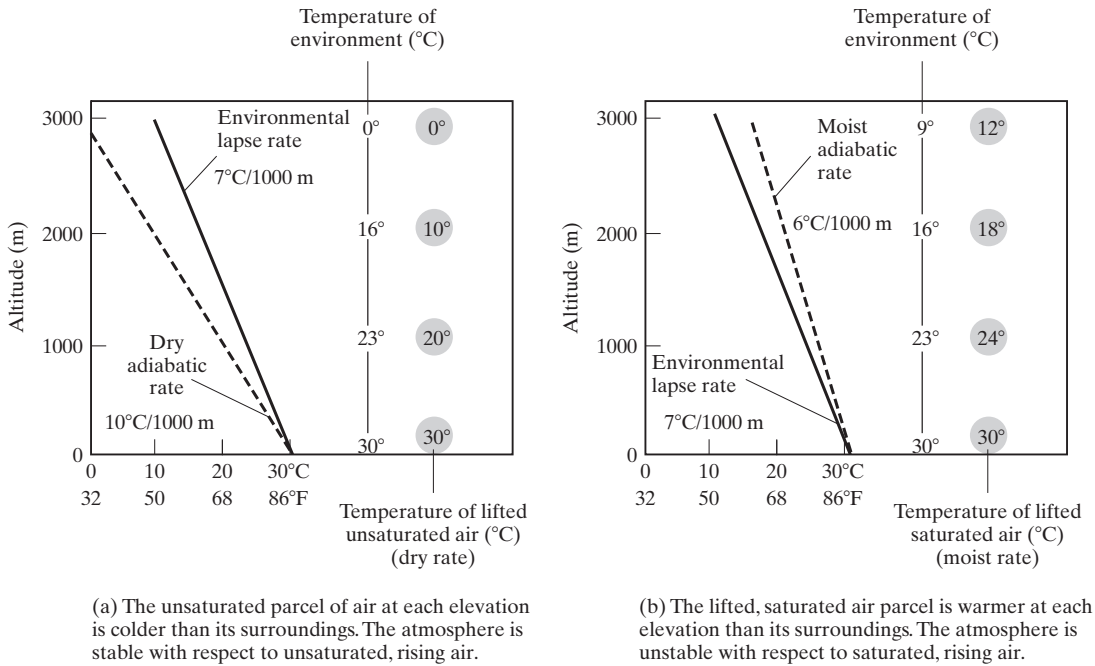
As moist, unsaturated air rises, the relative humidity increases, and at some elevation saturation is reached and relative humidity becomes 100%. Further cooling of the air results in condensation of the moisture at a defined lifting condensation level (LCL). Latent heat of condensation is released, warming the air and lowering the atmospheric lapse rate, or the rate of temperature change with elevation. As discussed earlier, latent heat exchange is the major energy source that fuels tropical cyclones and hurricanes. It has also been observed that a relation does not necessarily exist between the amount of water vapor and the resulting precipitation over a region. Thus, condensation can occur in cloud formations without the production of precipitation at the ground surface.

Meteorologists use the moisture relationships and the latent-heat concepts to obtain pressure–temperature relationships for cooling of rising moist air in the atmosphere. The rate of temperature change with elevation in the atmosphere is called the **adiabatic lapse rate**. The dry adiabatic lapse rate (DALR) is 9.8°C per km and assumes no phase changes of water. The average ambient lapse rate is about 6.5°C per km, but varies with moisture conditions. An unstable atmosphere is one in which the ambient lapse rate is greater than the DALR. A stable atmosphere is one in which the ambient lapse rate is less than the DALR, and an air parcel tends to cool faster than the environment as it rises vertically. Figure 1–8 shows the variation of lapse rates and different types of atmospheric conditions, with an unstable atmosphere being very conducive to lifting of moist air and to formation of severe weather and/or rainfall conditions.

Mechanisms of Precipitation Formation

Precipitation is the primary input to the hydrologic cycle, whether in the form of rainfall, snow, or hail, and is generally derived from atmospheric moisture. In order for precipitation to occur at the earth's surface,

1. a moisture source must be available,
2. moist air must undergo lifting and resultant cooling,

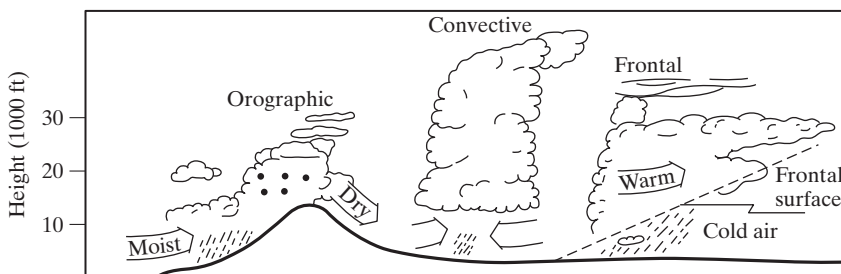
**Figure 1-8**

Variation of lapse rates and different types of atmospheric conditions. When air is colder than its surroundings and rises, it results in what is called a stable atmosphere. On the other hand, when warmer air rises, it results in an unstable atmosphere (greater movement of air).

3. a phase change must occur with resulting condensation onto small nuclei in the air,
4. droplets must grow large enough to overcome drag and evaporation to reach the ground.

Precipitation is often classified according to conditions that generate vertical air motion. Figure 1-9 shows the three main mechanisms:

1. **convective**, due to intense heating of air at the ground, which leads to expansion and vertical rise of air;

**Figure 1-9**

The three different precipitation lifting mechanisms that result when air at different temperatures meet in different topographies.

2. **cyclonic**, associated with the movement of large air-mass systems, as in the case of warm or cold fronts; and
3. **orographic**, due to mechanical lifting of moist air masses over the windward side of mountain ranges.

Orographic precipitation is caused by mechanical lifting of moist air over mountain ranges. Notable examples include the western side of the Cascade Mountains, the western side of the Andes in Chile, and the western coast of Norway. On the lee side of mountain barriers are dry areas, called rain shadows, since most of the moisture is dropped as rain or snow over the mountain ranges. Good examples of rain shadows can be found east of the Cascades in Washington and Oregon and east of the Sierra Nevada range in California (see Fig. 1–10).

Condensation of water vapor into cloud droplets occurs due to cooling of moist air to a temperature below the saturation point for water vapor. This is most commonly achieved through vertical lifting to levels where pressure and temperature are lower. A large portion of the atmospheric mass lies within 18,000 ft of the surface and contains most of the clouds and moisture. Condensation can be caused by (1) adiabatic cooling (no heat loss to surroundings), (2) mixing of air masses having different temperatures, (3) cooling by advection of cold air masses, and (4) cooling by radiation. Adiabatic cooling is by far the most important producer of appreciable precipitation.

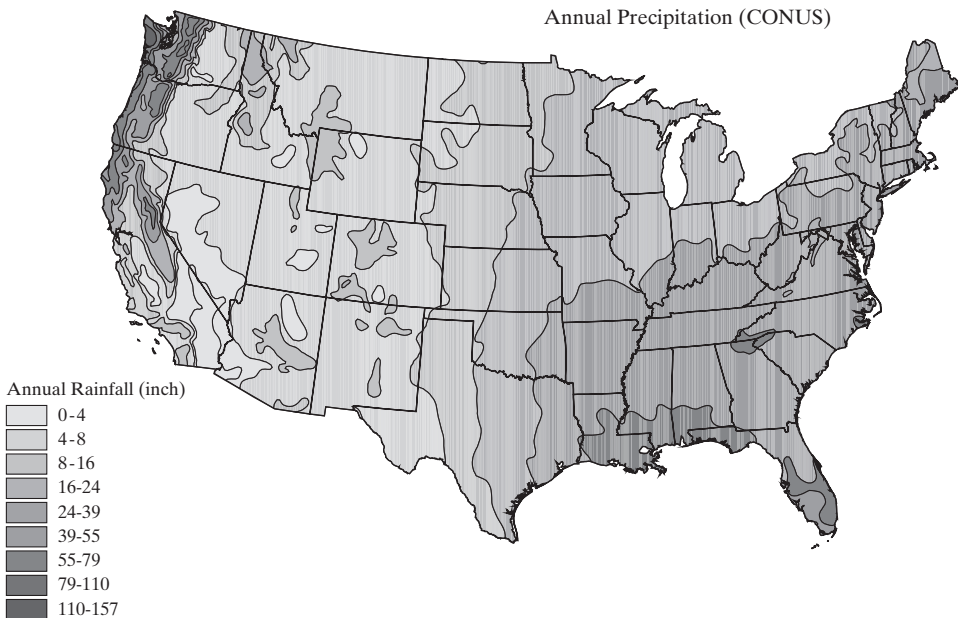


Figure 1–10

Distribution of average annual precipitation across the United States.

Dew, frost, and fog are minor producers of precipitation caused primarily by advective and radiative cooling.

Small condensation nuclei must be present for the formation of cloud droplets. Such nuclei come from many sources, such as ocean salt, dust from clay soils, industrial combustion products, and volcanoes, and they range in size from $0.1\ \mu$ to $10\ \mu$. Cloud droplets originally average 0.01 mm in diameter, and it is only when they exceed 0.5 mm that significant precipitation occurs. It may take hours for a small raindrop (1 mm) to grow on a condensation nucleus. As vapor-laden air rises, it cools as it expands; and as saturation occurs, water vapor begins to condense on the most active nuclei. The principal mechanism for the supply of water to the growing droplet in early stages is diffusion of water-vapor molecules down the vapor-pressure gradient toward the droplet surface. As the droplets increase in mass, they begin to move relative to the overall cloud. However, other processes must support the growth of droplets of sufficient size (0.5–3.0 mm) to overcome air resistance and to fall as precipitation. These include the coalescence process and the ice-crystal process.

The **coalescence process** is considered dominant in summer shower precipitation. As water droplets fall, the smaller ones are overtaken by larger ones, and droplet size is increased through collision. This can produce significant precipitation, especially in warm cumulus clouds in tropical regions. The **ice-crystal process** attracts condensation on freezing nuclei because of lower vapor pressures. The ice crystals grow in size through contact with other particles, and collisions cause snowflakes to form. Snowflakes may change into rain droplets after entering air in which the temperature is above freezing. Snowfall and snowmelt processes are presented in detail in Chapter 2.

Point Measurement

The main source of moisture for annual rainfall totals is evaporation from the oceans; thus, precipitation tends to be heavier near the coastlines, with distortion due to orographic effects—that is, effects of changes in elevation over mountain ranges. In general, amount and frequency of precipitation is greater on the windward side of mountain barriers (the western side for the United States) and less on the lee side (eastern side), also shown in Figure 1–10. Considerable amounts of precipitation data are available from the NWS, the USGS, and various local governmental agencies. A number of useful websites for precipitation data are listed in Appendix E and on the textbook website <http://hydrology.rice.edu>. Interpretation of national networks of rainfall data shows extreme variability in space and time, as can be seen in annual and monthly variations in Figures 1–10 and 1–11, respectively.

Time variation of precipitation occurs seasonally or within a single storm, and distributions vary with storm type, intensity, duration, and time

1.3 PRECIPITATION

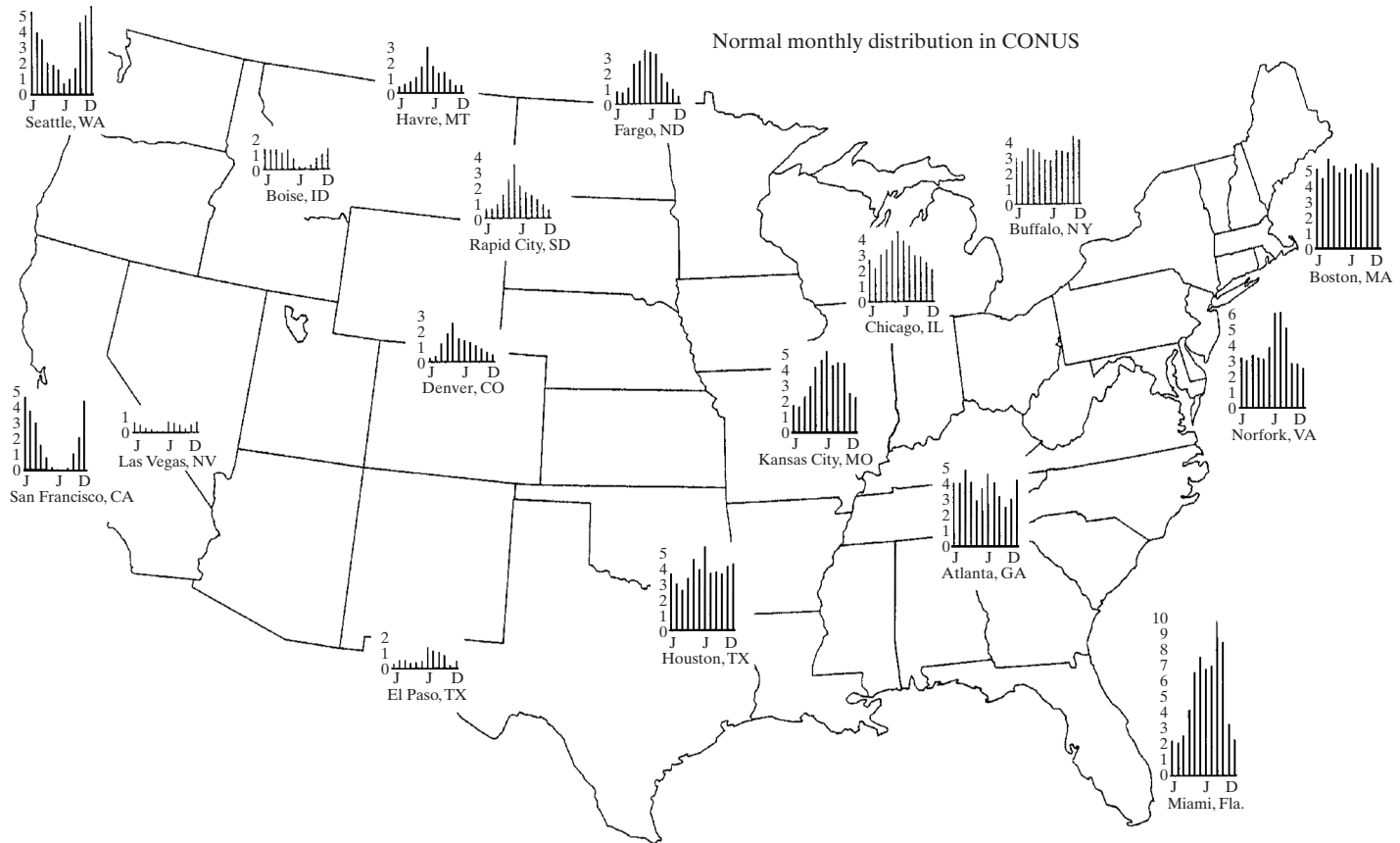


Figure 1-11

Normal monthly distribution of precipitation in different U.S. cities (in.) (1 in. = 25.4 mm). (U.S. Environmental Data Services.)

Table 1–2 Maximum Recorded Rainfall across the United States (in.)

	Duration		
	1 hr	6 hr	24 hr
San Francisco, CA	1.07	2.34	4.67
Portland, OR	1.31	—	7.66
Denver, CO	2.20	2.91	6.53
St. Louis, MO	3.47	5.82	8.78
New Orleans, LA	4.71	8.62	14.01
Alvin, TX (near Houston)	4.00	15.67	43.00
New York, NY	2.97	4.44	9.55
Miami, FL	4.53	10.64	15.10

of year. Prevailing winds and relative temperature of land and proximity of bordering ocean have an effect. One interesting statistic is the maximum recorded rainfall that can occur at a single gage. These data are shown for eight major U.S. cities in Table 1–2. The highest value for 24-hr rainfall in the United States was 43 in. (1092 mm) in Alvin near Houston, TX, indicating the impact of severe storms and hurricanes near coastal areas. World precipitation records, shown in Table 1–3, clearly indicate the effect of proximity to major oceans, as in the case of India.

Seasonal or monthly distributions for the United States are shown in Figure 1–11, where it is clear that areas such as Florida, California, and the Pacific Northwest have significant seasonal rainfall patterns compared to most areas in the country and along the eastern seaboard. Also, the west and southwest are significantly drier than the east or northwest. But the values shown are deceptive in that high-intensity thunderstorms or hurricanes can produce 15 to 30 in. of rainfall in a matter of days along the Gulf and Atlantic coasts. For example, Oregon and Washington receive most of their rainfall in the winter from fronts that move across the area, whereas in Florida thunderstorms and hurricanes produce large summer totals. Southern California, where most of the population resides,

Table 1–3 World Record Rainfalls

Duration	in.	mm	Location
1 min	1.50	38	Barot, Guadeloupe
15 min	7.80	198	Plumb Point, Jamaica
1 hr	15.80	401	Shangdi, China
12 hr	52.76	1,340	Belouve, Reunion
24 hr	72.40	1,840	Cherrapunji, India
2 days	98.42	2,500	Ciliaos, Reunion
15 days	253.60	6,443	Commerson, La Reunion
1 mo	366.14	9,300	Cherrapunji, India
1 yr	1041.78	26,461	Cherrapunji, India

gets significantly less rainfall than the northern part. This difference in available water led to the building of the California Water Project, which transports water hundreds of miles from the reservoirs in the north to the Los Angeles area.

Hourly or even more detailed variations of rainfall are often important for planning water resource projects, especially urban drainage systems. Figure 1–12 shows the cumulative rainfall for a major flood in the Houston area from 1979. Areal rainfalls such as shown in Figure 1–13 from T.S. Allison, which are useful in urban hydrologic studies. The intensity and duration of rainfall events and spatial variations are important in determining the hydrologic response for a watershed. Such data are available only

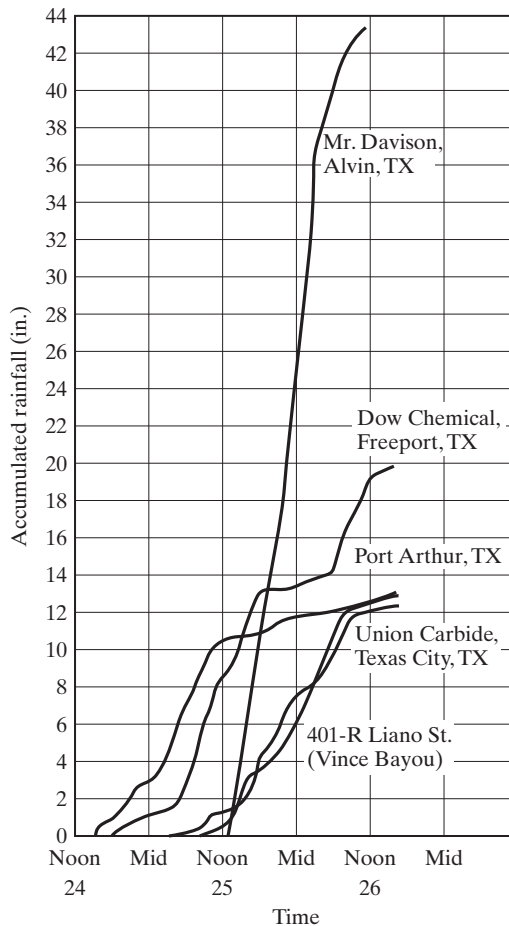


Figure 1–12

Accumulated rainfall for the July 1979 storm event near Houston, TX.

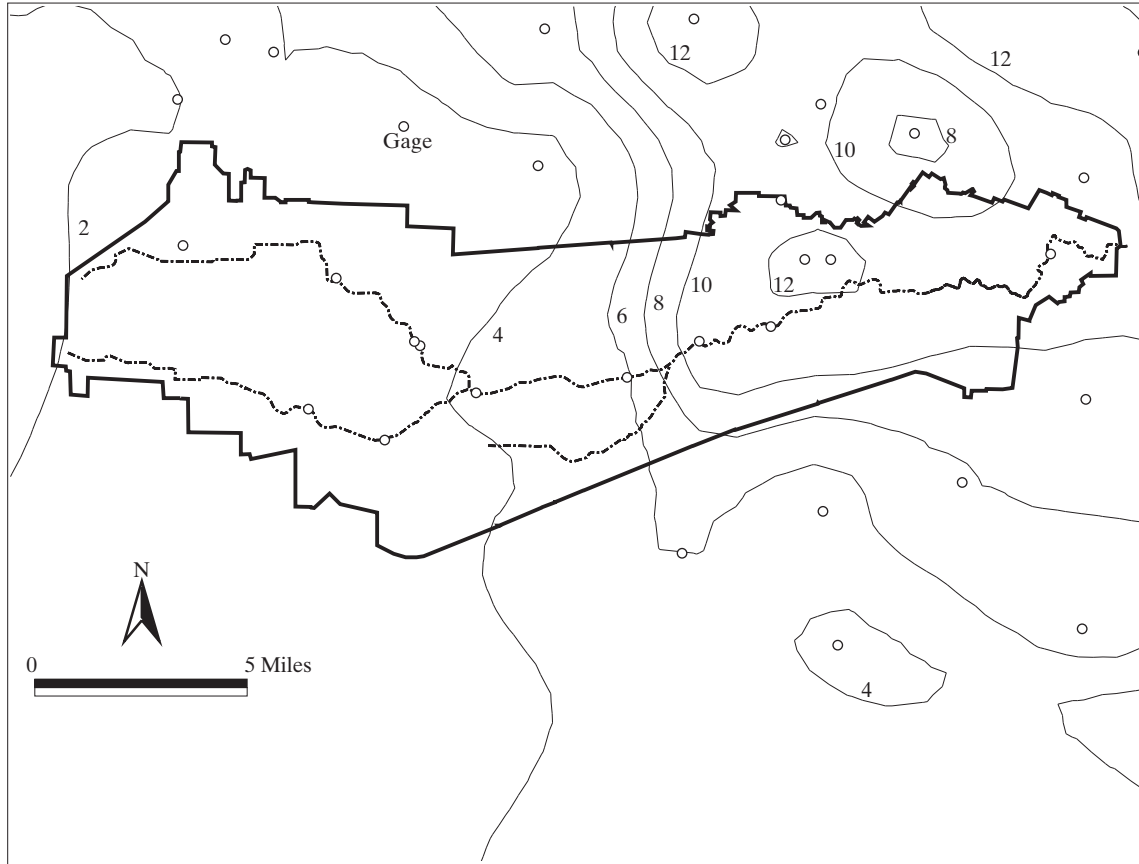


Figure 1-13

T.S. Allison 9-hr rainfall gage contours from Brays Bayou watershed in Houston, TX.

from sophisticated rainfall recording networks, usually located in larger urban areas and along major river basins. Rainfall gage networks are maintained by the NWS, the USGS, and local county flood control districts and utilities. An excellent source of rainfall data is now available on specific websites, such as National Climatic Data Center (NCDC) and NWS (see Appendix E).

Rainfall gages may be of the recording (Fig. 1–14a) or non-recording type, but recording gages are required if the time distribution of rainfall is desired, as is often the case for urban drainage or flood control works. The recording gage operates from a small tipping bucket that records on a data logger, every 0.1 or 0.01 in. of rainfall (or 0.1 or 1 mm in Canada). The data are displayed in a form shown in Figure 1–12 as a **cumulative mass curve** and can be readily interpreted for total volume and intensity variations. Observers usually report daily or 12-hr amounts

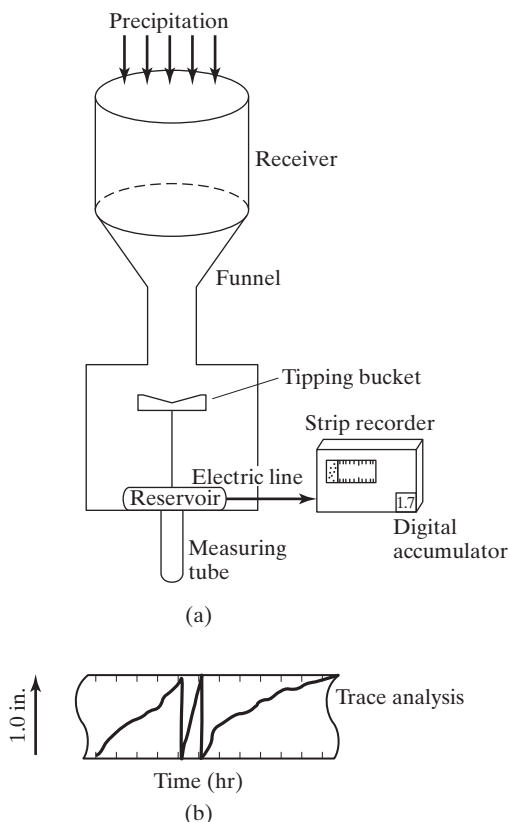


Figure 1–14a

Recording tipping bucket gage. Trace returns to zero after each inch of rainfall. The slope of the trace registers intensity (in./hr).



Figure 1-14b
Typical recording gage.

of rainfall (in. or mm) for non-recording gages, providing little information on intensity. A typical rainfall and stream gage with telemetry is shown in Figure 1-14b.

Point rainfall can be plotted as accumulated total rainfall or as rainfall intensity vs. time at a particular gage. The first plot is referred to as a cumulative mass curve (Fig. 1-12), which can be analyzed for a variety of storms to determine the frequency and character of rainfall at a given site. A **hyetograph** is a plot of rainfall intensity (in./hr) vs. time, and one is depicted in Example 1-2 along with cumulative mass curves for total rainfall. Hyetographs are often used as input to hydrologic computer models for predicting watershed response to input rainfall.

HYETOGRAPHS AND CUMULATIVE PRECIPITATION

EXAMPLE 1-2

Table E1-2 is a record of precipitation from a recording gage for a storm in Texas, for the period between midnight and 11:15 A.M. on the same day in increments of 0.25 hr. For the data given, develop the rainfall hyetographs and mass curves. Find the maximum-intensity rainfall for the gage in in./hr.

Table E1-2 Rainfall Data from a Recording Gage

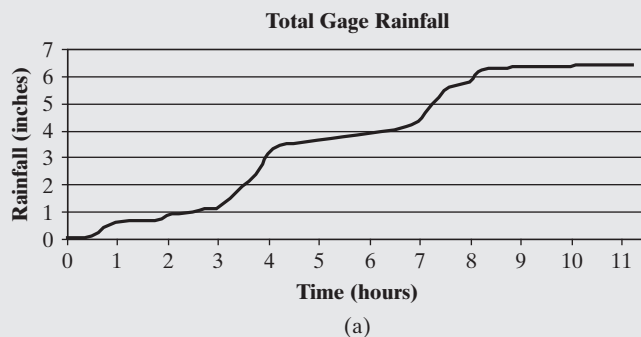
Time (hr)	Gage Rainfall (in.)	Gage Intensity (in./hr)	Time (hr)	Gage Rainfall (in.)	Gage Intensity (in./hr)
0	0	0	5.75	3.78	0.24
0.25	0.02	0.08	6	3.84	0.24
0.5	0.07	0.2	6.25	3.9	0.24
0.75	0.4	1.32	6.5	3.95	0.2
1.0	0.55	0.6	6.75	4.1	0.6
1.25	0.6	0.2	7.0	4.3	0.8
1.5	0.62	0.08	7.25	4.93	2.52
1.75	0.62	0	7.5	5.4	1.88
2.0	0.82	0.8	7.75	5.61	0.84
2.25	0.88	0.24	8.0	5.77	0.64
2.5	0.92	0.16	8.25	6.17	1.6
2.75	1.06	0.56	8.5	6.22	0.2
3.0	1.1	0.16	8.75	6.27	0.2
3.25	1.47	1.48	9.0	6.29	0.08
3.5	1.87	1.6	9.25	6.3	0.04
3.75	2.32	1.8	9.5	6.31	0.04
4.0	3.1	3.12	9.75	6.32	0.04
4.25	3.4	1.2	10.0	6.33	0.04
4.5	3.48	0.32	10.25	6.34	0.04
4.75	3.54	0.24	10.5	6.35	0.04
5.0	3.62	0.32	10.75	6.36	0.04
5.25	3.68	0.24	11.0	6.37	0.04
5.5	3.72	0.16	11.25	6.38	0.04

SOLUTION

To plot the hyetograph for a gage, we subtract the measurement for each time period from that of the previous time period, and divide by the time step to compute intensity as shown in the table. Because the data are given as a cumulative reading, the mass curves are simply a plot of the data as given (see Fig. E1-2).

Figure E1-2

Total gage rainfall and gage rainfall intensity.



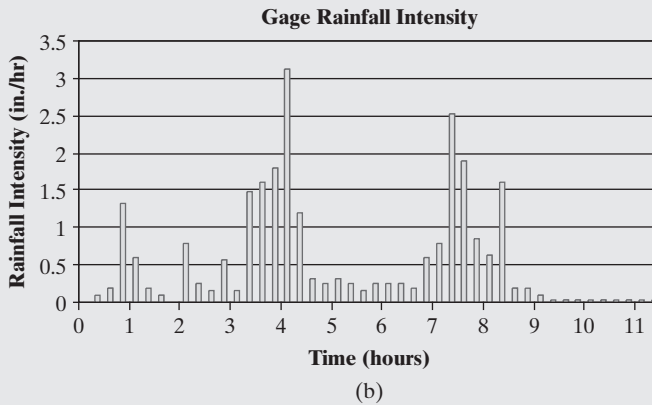


Figure E1-2b

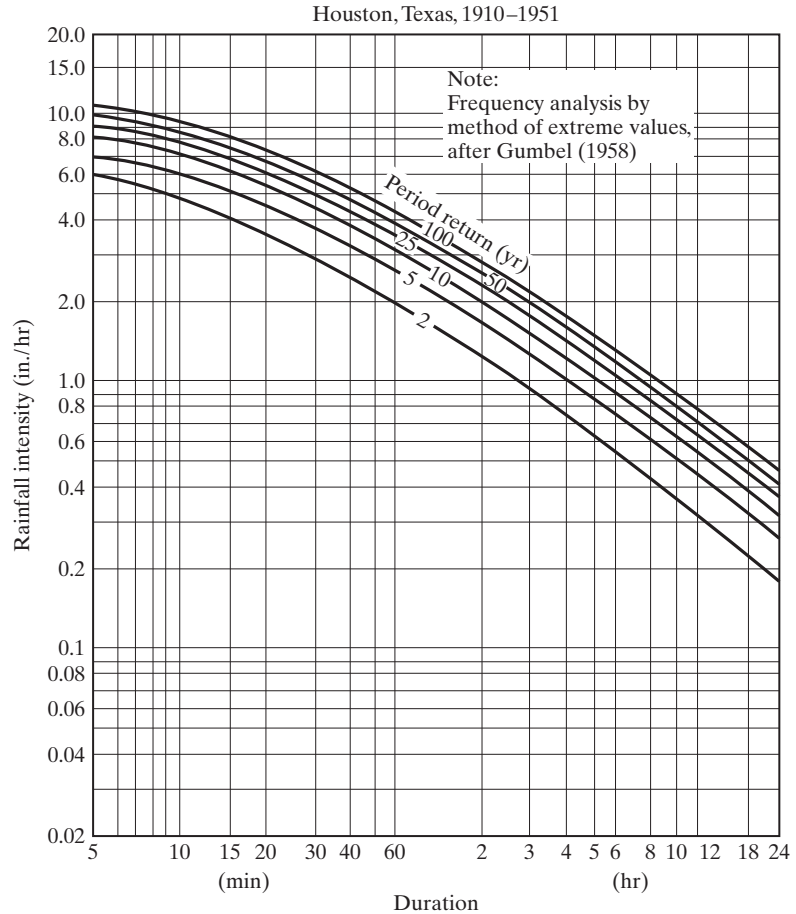
The maximum intensity for the gage occurred around 4:00 A.M.

$$\frac{(3.1 - 2.32) \text{ in.}}{0.25 \text{ hr}} = 3.12 \text{ in./hr}$$

This maximum intensity appears as the tallest bar on the hyetograph and as the region of greatest slope on the cumulative precipitation curve. This illustrates that the mass curve is the integral of the hyetograph, as, in probability theory, the cumulative distribution function is the integral of the probability density function. Note that the gage had two distinct periods of intense rainfall. These periods of rainfall intensity have the capacity to produce significant runoff and flooding.

Statistical methods (Chapter 3) can be applied to a long time series of rainfall data. For example, rainfalls of various duration ranging from 5 min to 24 hr can be analyzed to develop an estimate of, for example, the 100-yr frequency event. These data are fitted with a contour line to form one of the curves on the **intensity–duration–frequency** (IDF) curves in Figure 1–15. Other IDF probability lines are derived in a similar fashion for the 2-yr, 5-yr, 10-yr, 25-yr, and 50-yr design rainfalls. It should be noted that IDF curves do not represent the time history of actual storms. Data points on an IDF curve are usually derived from many segments of longer storms, and the values extrapolated by frequency analysis. It can be seen that the intensity of rainfall tends to decrease with increasing duration of rainfall for each of the IDF curves. Instead of analyzing historical rainfall time series, the IDF curves can be used to derive design rainfall events, such as the 10-yr, 2-hr storm, which equals 2.0 in./hr, or the 10-yr, 24-hr storm, which equals 0.3 in./hr or 7.2 in. in 24 hr. One of the homework problems indicates how this procedure is carried out. Such design storms are often used as input to a hydrologic model for drainage design or flood analysis (see Chapters 5 and 6).

Figure 1-15
Intensity-duration
frequency curves for
Houston, TX.



It is sometimes necessary to estimate point rainfall at a given location from recorded values at surrounding sites. The NWS (1972) has developed a method for this based on a weighted average of surrounding values. The weights are reciprocals of the sum of squares of distances D , measured from the point of interest. Thus, for four rain gages where one of them did not record (P1), one would estimate D_2 , D_3 , and D_4 , distances from the non-functional gage. Then the estimate for P1, based on measured values of P2, P3, and P4, would be given by Equation 1-8 below with the weights determined by the inverse square of the distance away from P1.

$$D^2 = x^2 + y^2, \quad (1-6)$$

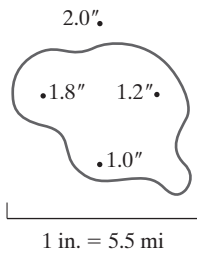
$$W = 1/D^2 = \text{weight}, \quad (1-7)$$

$$\text{rainfall estimate} = \frac{\sum_i P_i W_i}{\sum_i W_i}. \quad (1-8)$$

Areal Precipitation

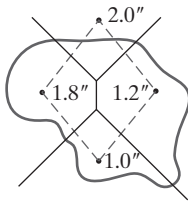
Predicting watershed response to a given precipitation event often requires knowledge of the average rainfall that occurs over a watershed area in a specified duration. The average depth of precipitation over a specific watershed area is more accurately estimated for an area that is well monitored. Three basic methods exist to derive areally averaged values from point rainfall data: the arithmetic mean, the Thiessen polygon method, and the isohyetal method. Radar-based estimates of rainfall provide an interesting alternative for areas where rainfall gages may be lacking, and these methods are described in Chapter 11.

The simplest method is an **arithmetic mean** of point rainfalls from available gages [Fig. 1-16a]. This method is satisfactory if the gages are uniformly distributed and individual variations are not far from the mean rainfall. The method is not particularly accurate for larger areas where rainfall distribution is variable.



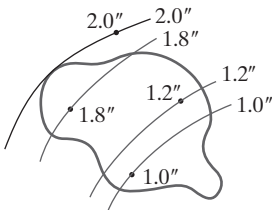
(a) Arithmetic mean

$$\frac{1.8 + 1.2 + 1.0}{3} = 1.33 \text{ in.}$$



(b) Thiessen polygon method

P_i (in.)	A_i (mi ²)	A_i/A_r	$(P_i)(A_i/A_r)$ (in.)
2.0	1.5	0.064	0.13
1.8	7.2	0.305	0.55
1.2	5.1	0.216	0.26
1.0	9.8	0.415	0.42
$\Sigma =$	23.6	1.000	1.35 in.



(c) Isohyetal method

Isohyet (in.)	A (mi ²)	P_{av} (in.)	V (in.-mi ²)
2.0	5.1	1.9	9.69
1.8	9.8	1.5	14.7
1.2	3.1	1.1	3.41
1.0	5.6	0.5*	2.8
	23.6		30.6

$$\text{Average rainfall} = 30.6/23.6 = 1.30 \text{ in.}$$

* Estimated

Figure 1-16

The different rainfall averaging methods can produce different results for the same watershed.

The **Thiessen polygon method** [Fig. 1–16(b)] allows for areal weighting of rainfall from each gage. Such a polygon is the locus of points closer to the given gage than to any other. Connecting lines are drawn between stations located on a map. Perpendicular bisectors are drawn to form polygons around each gage, and the ratio of the area of each polygon A_i within the watershed boundary to the total area A_T is used to weigh each station's rainfall. The method is unique for each gage network and does not allow for orographic effects (those due to elevation changes), but it is probably the most widely used of the three available methods.

The **isohyetal method** [Fig. 1–16(c)] involves drawing contours of equal precipitation (isohyets) and is the most accurate method. However, an extensive gage network is required to draw isohyets accurately. The rainfall calculation is based on finding the average rainfall between each pair of contours, multiplying by the area between them, totaling these products, and dividing by the total area. The isohyetal method can include orographic effects and storm morphology and can represent an accurate map of the rainfall pattern, as shown for T. S. Allison for a watershed in Houston, TX (see Fig. 1–13).

EXAMPLE 1–3

RAINFALL AVERAGING METHODS

A watershed covering 28.16 mi^2 has a system of seven rainfall gages, as shown on the map in Figure E1–3a. Using the total storm rainfall depths given in the accompanying table, determine the average rainfall over the watershed using (a) arithmetic averaging and (b) the Thiessen polygon method.

Gage	Rainfall (in.)
A	5.13
B	6.74
C	9.00
D	6.01
E	5.56
F	4.98
G	4.55

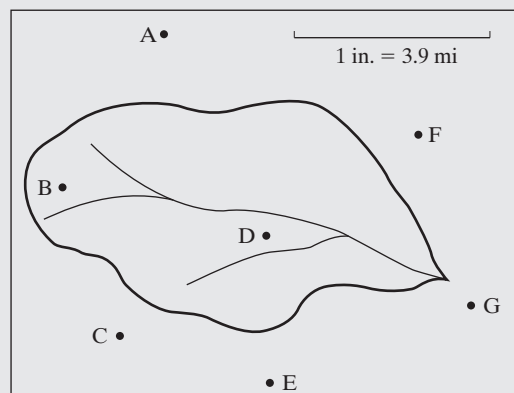


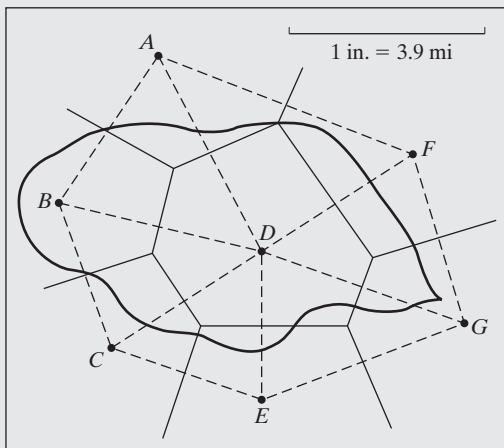
Figure E1–3a

Placement of rainfall gages to record rainfall and resulting outflow in the watershed.

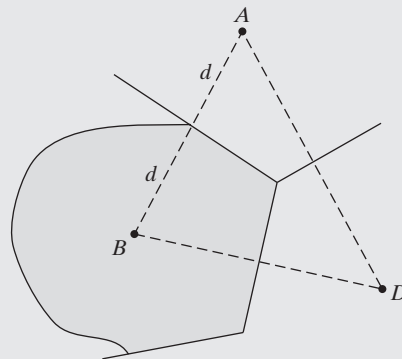
- (a) For the arithmetic averaging method, only the gages within the watershed are used—in this example the gages B and D. Thus, the arithmetic average is

$$(6.74 + 6.01)/2 = 6.38 \text{ in.}$$

- (b) The first step in the Thiessen polygon method is to connect all nearby rain gages by straight lines. The result is a system of triangles, as shown by the dashed lines in Figure E1–3b. Next, we construct perpendicular bisectors of the dashed lines in Figure E1–3c. The bisectors meet at a common point inside or outside the triangle. The resulting polygons around each rainfall gage are known as the Thiessen polygons.

SOLUTION**Figure E1–3b**

Resulting bisectors of rainfall gages to find rainfall in the watershed.

**Figure E1–3c**

Bisectors for the Thiessen polygons.

The area of each polygon within the watershed boundary is measured using a map tool or GIS, or by counting squares on graph paper, and each individual area is divided by the total watershed area and multiplied by the depth of rainfall, measured at its corresponding gage. The sum of fraction area times rainfall for all the gages gives the average rainfall over the watershed. These computations, easily carried out in Microsoft Excel, are shown in the following table. A perpendicular bisector separates the triangle legs into two equal length segments. It intersects the leg at a 90-degree angle. The Thiessen polygons that weigh each rain gage are created by the solid perpendicular bisector lines and the boundary of the watershed.

Thus, the Thiessen polygon method (see Table E1–3) gives an average rainfall over the basin of

6.13 in., compared to 6.38 in. above

Table E1–3

Gage	P_i (in.)	A_i (mi ²)	A_i/A_t	$(P_i)(A_i/A_t)$ (in.)
A	5.13	1.74	0.062	0.32
B	6.74	6.70	0.238	1.60
C	9.00	1.77	0.063	0.57
D	6.01	13.02	0.463	2.78
E	5.56	0.83	0.029	0.16
F	4.98	2.68	0.095	0.47
G	4.55	1.42	0.050	0.23
		28.16	1.000	6.13

Radar-Based Precipitation

Advances in weather radar (called **NEXRAD** for next-generation radar) in the early 1990s greatly improved our ability to determine rainfall rates over watershed areas. NEXRAD reflects off raindrops in the atmosphere to estimate rainfall rates in time and space. NEXRAD is a 10-cm-wave-length radar that records reflectivity, radial velocity, and spectrum width of reflected signals. A more complete description of radar data products and processing may be found in Chapter 11 and in Crum and Alberty (1993), Klazura and Imy (1993), Smith et al. (1996), Fulton et al. (1998), and Vieux (2004).

Until the advent of the NEXRAD system nationwide, gaging stations were the only source of rainfall data for hydrologic modeling and flood prediction. Radar data can be translated from their original radial coordinates from the source radar into a gridded coordinate system with 1.0-km² resolution grids. Recent efforts have been successful in measuring rainfall rates and cumulative totals using radar technology developed and implemented in the 1990s (Vieux and Bedient, 1998; Bedient et al., 2000, 2003; Vieux, 2004). Figure 1–17 depicts the type of radar rainfall information available from NEXRAD radar systems every 5–6 minutes for a storm event in Louisiana. Chapters 10 and 11 present the background and details for using radar data to support hydrologic prediction from models and for associated flood alert systems.

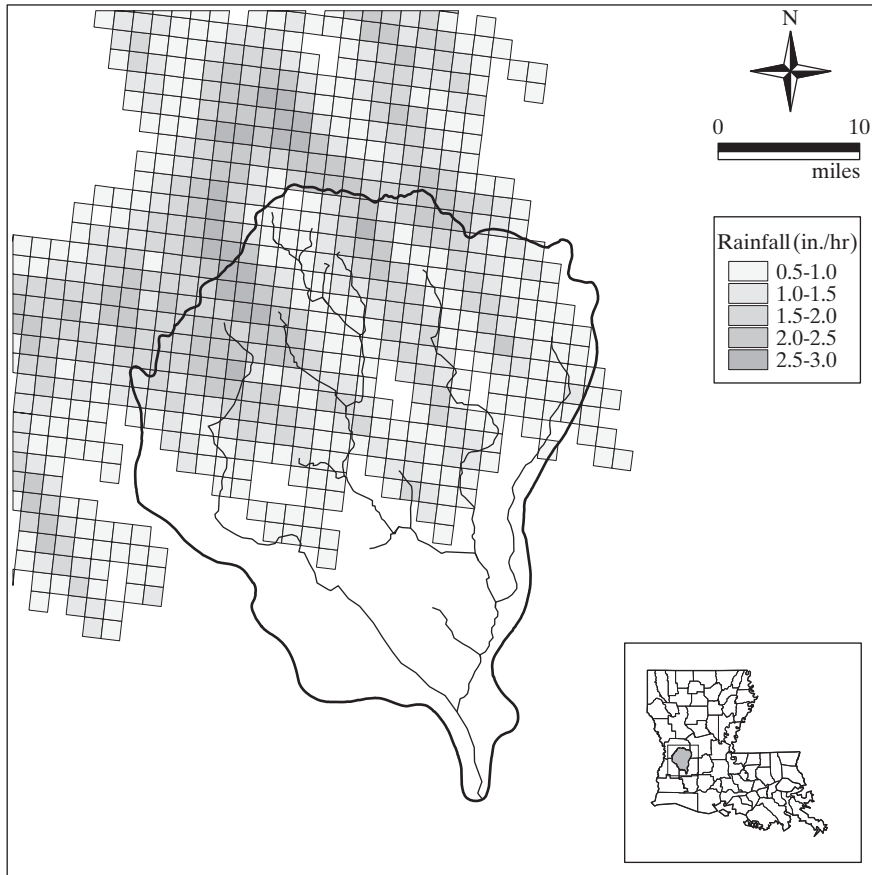


Figure 1-17

Typical NEXRAD rainfall data for a watershed located in central Louisiana.

The hydrologic cycle is a very complex series of processes (Fig. 1-1b), but under certain well-defined conditions the response of a watershed to rainfall, infiltration, and evaporation can be calculated if simple assumptions can be made. A **watershed** is a contiguous area that drains to an outlet, such that precipitation that falls within the watershed runs off through that single outlet (the term **catchment** is sometimes used synonymously for just the surface portion of the watershed). For example, if the rainfall rate over a watershed area is less than the rate of infiltration into soil and if there is ample storage in soil moisture, then direct runoff from the surface and resulting streamflow will be zero. If, on the other hand, antecedent or previous rainfall has filled soil storage and if the rainfall rate is so large that infiltration and evaporation can be neglected, then the volume of surface runoff will be equal to the volume of rainfall. In most cases, however, the conditions fall somewhere

1.4 THE HYDROLOGIC CYCLE

between these limitations, and we must carefully measure or calculate more than one component of the cycle to predict watershed response. The watershed is the basic hydrologic unit within which all measurements, calculations, and predictions are made in hydrology (see Fig. 1-13).

The Water Balance

The basic components of the hydrologic cycle include precipitation, evaporation, evapotranspiration, infiltration, overland flow, streamflow, and ground water flow (Fig. 1-1a). The movement of water (rainfall and runoff) through various phases of the hydrologic cycle varies greatly in time and space, giving rise to extremes of floods or droughts. The magnitude and the frequency of occurrence of these extremes are of great interest to the engineering hydrologist from a design and operations standpoint. In some cases, it is possible to perform a water budget calculation in order to predict changes in storage to be expected based on inputs and outputs from the system.

For any hydrologic system, a water budget can be developed to account for various flow pathways and storage components. The hydrologic continuity equation for any system is

$$I - Q = \frac{dS}{dt}, \quad (1-9)$$

where

I = inflow in L^3/t ,

Q = outflow in L^3/t ,

dS/dt = change in storage per time in L^3/t .

The simplest system is an impervious inclined plane, confined on all four sides with a single outlet. A small urban parking lot follows such a model, and as rainfall accumulates on the surface, the surface detention, or storage, slowly increases and eventually becomes outflow from the system. Neglecting evaporation for the period of input, and assuming a long rainfall time period, all input rainfall eventually becomes outflow from the area, but delayed somewhat in time. The difference between inflow to the parking lot and outflow at any time represents the change in storage [Eq. (1-9)]. Thus, the total storage volume that is eventually released from the area is equal to the accumulated difference in inflow volume and outflow volume, or $\int (I - Q) \Delta t$.

The same concept can be applied to small basins or large watersheds. Note that urban watersheds include both natural and man-made elements. For a given time period, a conceptual mathematical model for the budget for the urban hydrologic cycle shown in Figure 1-18 would become, in units of depth (in. or mm) over the basin,

$$P - R - G - E - T = \Delta S, \quad (1-10)$$

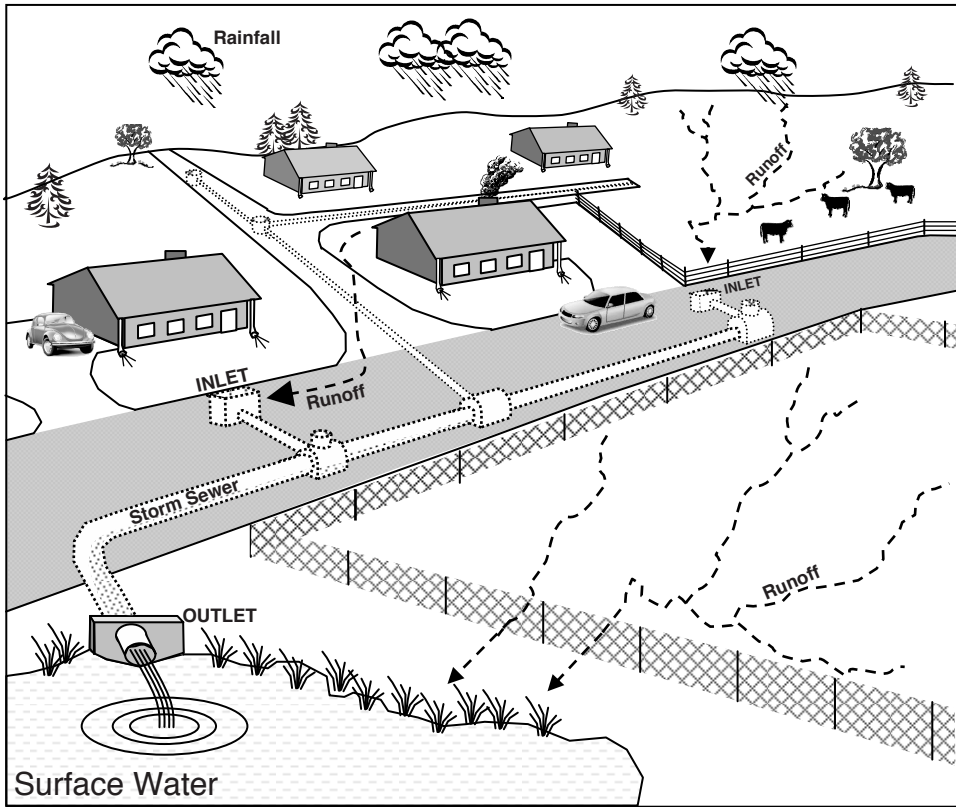


Figure 1-18

Urban hydrologic cycle. A combination of runoff from natural surroundings and man-made drainage systems, these runoffs come together at a single outlet.

where

- P = precipitation,
- R = surface runoff,
- G = ground water flow,
- E = evaporation,
- T = transpiration,
- ΔS = change in storage in a specified time period.

A **runoff coefficient** can be defined as the ratio R/P for any watershed. Note that infiltration I is a loss from the surface system and a gain to ground water, and thus cancels out of the overall budget above. Also, the units of inches (or mm) represent a volume of water when multiplied by the surface area of the watershed. If a water body receives inflow for a period of time, the change in water depth due to the inflow can be easily calculated. There are two ways of denoting the volume of water that is added, either as a flow

rate for a specified time or as a water depth across an area. The following equation results:

$$\text{volume} = (\text{flow rate})(\text{time}) = (\text{depth})(\text{watershed area}). \quad (1-11)$$

Typical units may be English or metric, as indicated below:

flow rate	cfs or m ³ /s
time	seconds, days months
depth	in. or mm
area	acres, sq mi, or sq km

To convert from a flow to a change in water depth, rearrange the equation above and multiply by necessary conversion factors:

$$\text{depth} = \frac{(\text{flow rate})(\text{time})(\text{conversion factor})}{\text{watershed area}}. \quad (1-12)$$

Conversion factors = (30 days/month) (24 hr/day) (3600 s/hr) (1 acre/43,560 ft) (12 in./ft). Note that **1.0 ac-inch = 1.008 cfs-hr**.

EXAMPLE 1-4

WATER BALANCE IN A LAKE

For a given month, a 300-acre lake has 15 cfs of inflow, 13 cfs of outflow, and a total storage increase of 16 ac-ft. A USGS gage next to the lake recorded a total of 1.3 in. precipitation for the lake for the month. Assuming that infiltration loss is insignificant for the lake, determine the evaporation loss, in inches, over the lake for the month.

SOLUTION

Solving the water balance for inflow I and outflow O in a lake gives, for evaporation,

$$\underset{\text{evaporation}}{E} = \underset{\text{inflow}}{I} - \underset{\text{outflow}}{O} + \underset{\text{precipitation}}{P} - \underset{\text{change in storage}}{\Delta S},$$

$$I = \frac{(15 \text{ ft}^3/\text{s})(\text{ac}/43,560 \text{ ft}^2)(12 \text{ in./ft})(3600 \text{ s/hr})(24 \text{ hr/day})(30 \text{ day/month})(1 \text{ month})}{300 \text{ ac}}$$

$$= 35.70 \text{ in.},$$

$$O = \frac{(13 \text{ ft}^3/\text{s})(\text{ac}/43,560 \text{ ft}^2)(12 \text{ in./ft})(3600 \text{ s/hr})(24 \text{ hr/day})(30 \text{ day/month})(1 \text{ month})}{300 \text{ ac}}$$

$$= 30.94 \text{ in.},$$

$$P = 1.3 \text{ in.},$$

$$\Delta S = \frac{(16 \text{ ac} - \text{ft})(12 \text{ in./ft})}{300 \text{ ac}} = 0.64 \text{ in.},$$

$$E = (35.70) - (30.94) + (1.3) - (0.64) \text{ in.},$$

$$E = 5.42 \text{ in.}$$

WATER BALANCE IN A SWIMMING POOL**EXAMPLE 1-5**

A swimming pool (20 ft $\times$ 20 ft $\times$ 5 ft) has a small leak at the bottom. You are given measurements of rainfall, evaporation, and water level on a daily basis for 10 days. As an engineer, use the water balance to determine the average daily leakage out of the swimming pool in ft³/day. Assume the pool is exactly 5 ft (60 inches) deep at the end of day 1.

Day	Evaporation (in.)	Rainfall (in.)	Measured Level (in.)
1	0.5		60
2	0	1.0	
3	0.5		
4	0	2.0	
5	0.5		
6	0.5		
7	0	4.0	
8	0.5		
9	0.5		
10	0.5		52

The water-balance equation becomes:

SOLUTION

$$\text{outflow} = \text{precipitation} - \text{evaporation} - \Delta \text{storage}$$

All values must be in the same units. Thus, the total change in storage is $52 - 60 = -8$ in. The precipitation is $1.0 + 2.0 + 4.0 = 7$ in. The evaporation is $(7)(0.5) = 3.5$ in.

Thus,

$$\text{outflow} = 7 - 3.5 - (-8),$$

$$\text{outflow} = 11.5 \text{ in.}$$

Outflow should be in ft³/day. The height change is distributed over the pool area.

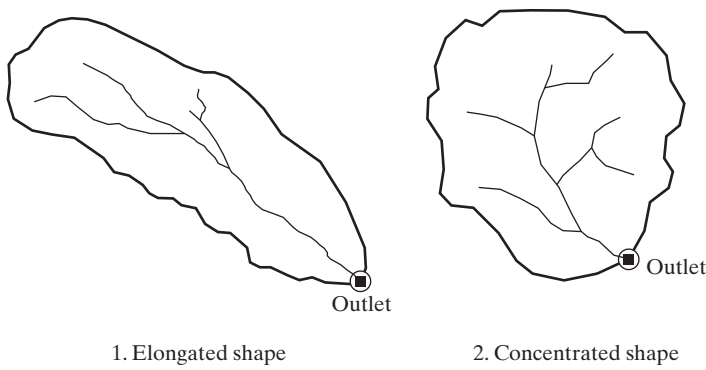
$$\text{outflow} = \frac{(11.5 \text{ in.})(1 \text{ ft}/12 \text{ in.})(20 \text{ ft})(20 \text{ ft})}{10 \text{ days}},$$

$$\text{outflow} = 38.3 \text{ ft}^3/\text{day}.$$

The Watershed

The watershed or basin area is an important physiographic property that determines the volume of runoff to be expected from a given rainfall event that falls over the area. Watershed areas vary in size from a few acres in an urban area to thousands of square miles for a major river basin. The **watershed divide** is the loci of points (the ridge line) that separates two

Figure 1–19a
Typical watershed area shapes. The difference in shape affects timing and peak flow of runoff to the outlet.



adjacent watersheds, which then drain into two different outlets. Figure 1–19a depicts several watershed areas that have been defined based on topographic or elevation data.

The topographic divide for a basin is usually drawn on a USGS map or quadrangle sheet (1:24,000 scale) or other topographic map by identifying high points and contours of constant elevation to determine directions of surface runoff. The area encompassed by the divide is the watershed area. Runoff originates at higher elevations and moves toward lower elevations in a direction perpendicular to the contour lines, as shown in Figure 1–19b. Note that the

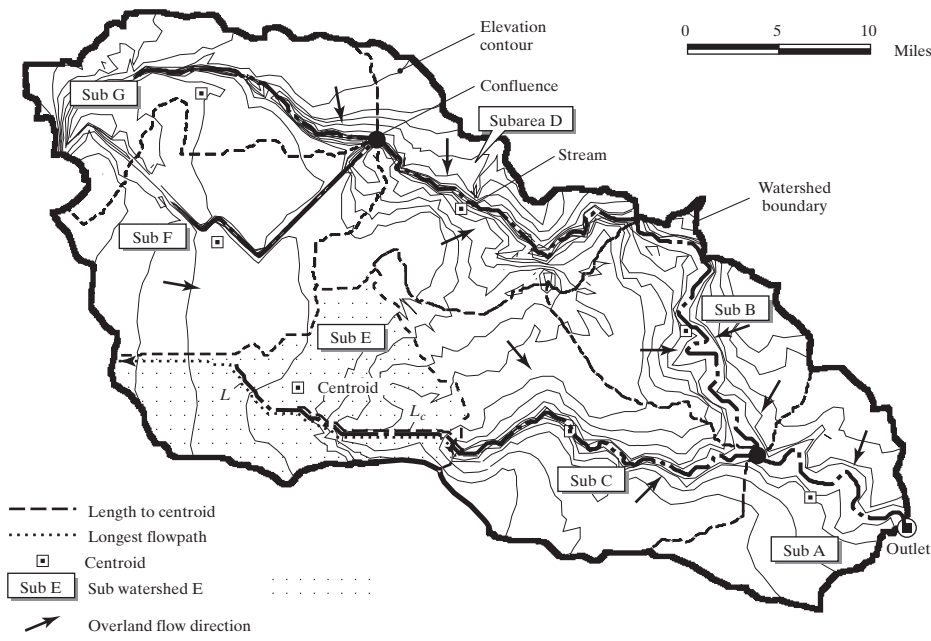


Figure 1–19b
Subwatershed delineation with overland flow direction and elevation contours.

arrows indicate the directions of flow in each subarea (A through G), and that flows generally move toward the nearest stream in a down-gradient direction.

In general, the larger the watershed area, the greater the surface runoff rate, the greater the overland flow rate, and the greater the streamflow rate. Formulas developed to relate peak flow to watershed area take the form $Q_p = cA^n$, where Q_p = peakflow, A = watershed area, and c and n are regression constants to be determined (Chapter 3). Peak flow is the product of a channel cross-sectional area and its average velocity at peak conditions. Watershed area is an important parameter that governs peak flow in most of the hydrologic prediction methods described later in this chapter and throughout the text.

Watershed relief is the elevation difference between two reference points within a watershed. Maximum relief is the difference between the highest point on the watershed divide and the watershed outlet. The longitudinal profile of the main channel is a plot of elevation vs. horizontal distance and is an indicator of channel gradient. Most streams, and especially rivers, show a decrease in channel gradient as one proceeds in a downstream direction (see Fig. 1–19b). This is due to the interaction of bottom friction and water depth. Channel gradients vary from about 0.1 (10%) for a mountain stream and as low as 0.0001 (.01%) for coastal areas.

Hydrologists are concerned with the amount of surface runoff generated in a watershed for a given rainfall pattern, and attempts have been made to analyze historical rainfall, infiltration, evaporation, and streamflow data to develop predictive relationships. When rainfall exceeds the infiltration rate at the surface, excess water begins to accumulate as surface storage in small depressions governed by surface topography. Eventually, the entire area is contributing to runoff at the outlet of a watershed.

The USGS as well as local flood control agencies are responsible for extensive hydrologic gaging networks within the United States, and data gathered on an hourly or daily basis can be plotted for a given watershed to relate rainfall to direct runoff for various time periods. Annual rainfall–runoff relationships remove seasonal effects and other storage effects, so that the relationship of rainfall minus losses vs. runoff can often be approximated by a linear regression line. The USGS developed a series of reports and relationships for predicting flood-peak discharges for urban and rural areas in the United States. Some of these are reviewed in Chapter 3.

Simple rainfall–runoff relationships should be used in water resources planning studies only where approximate water balances are required. A detailed knowledge of the magnitude and time distribution of both rainfall and runoff or streamflow is required for most flood control or floodplain studies, especially in urban watersheds. Many websites listed in Appendix E support USGS streamflow data from most of the stations in the United States.

1.5 SIMPLE RAINFALL– RUNOFF

One of the simplest rainfall–runoff formulas, which is often used for drainage design purposes in small watersheds and/or basins, is the **Rational Method** (Chapters 6 and 9), which allows for the prediction of peak flow Q_p (cfs) from the formula

$$Q_p = CiA \quad (1-13)$$

where

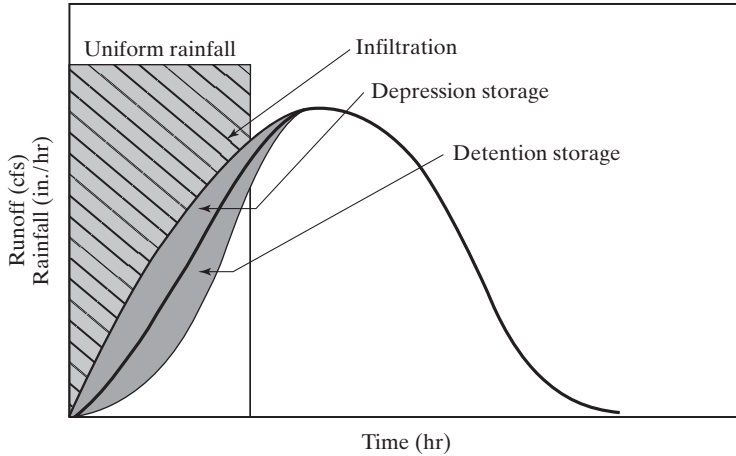
- C = runoff coefficient, variable with land use,
- i = intensity of rainfall of chosen frequency for a duration equal to time of concentration t_c (in./hr),
- t_c = equilibrium time for rainfall occurring at the most remote portion of the basin to contribute flow at the outlet (min or hr),
- A = area of watershed (acres).

The Rational Method is usually attributed to Kuichling (1889) and Lloyd-Davies (1906), but Mulvaney (1851) clearly outlined the procedure in a paper in Ireland. The underlying assumption is that a steady, uniform rainfall rate will produce maximum runoff when all parts of a watershed are contributing to outflow, a condition that is met after the time of concentration t_c has elapsed. Time of concentration t_c is defined as the time for a wave of water to propagate from the most distant point in the basin to the outlet (see Section 2.1). Runoff is assumed to reach a maximum when the rainfall intensity lasts as long as the t_c . The runoff coefficient is assumed constant during a storm event. More details can be found in later chapters and in McCuen (2005). The rational method is often used in small urban areas to design drainage systems, including pipe systems, culverts, and open channels. Chapters 6 and 9 present detailed discussion and examples of the rational method applied to small watershed areas. Larger watersheds (greater than a few square miles) usually require a consideration of the entire hydrograph because timing and storage issues become important, and the rational method is usually limited to basins less than a few hundred acres in size.

1.6 STREAMFLOW AND THE HYDROGRAPH

The hydrologic cycle is shown as a schematic in Figure 1–1, where precipitation P initially falls on the land surface, and it may distribute to fill **depression storage**, infiltrate to become soil moisture and shallow ground water, or travel as interflow to a receiving stream. Evaporation E is often a small component of a specific storm event, since it is minimal as rain is actually falling, and is usually only a factor in longer-term water balances. Depression storage capacity is usually satisfied early in storm passage, followed by infiltration capacity into the soil (see Fig. 1–20). Eventually, overland flow and surface runoff commence after soil storage and depression storage are satisfied. Overland flow then quickly moves down-gradient toward the nearest small rivulet or channel, which then flows into the next larger stream, and eventually reaches the main stream channel as open-channel flow.

The **hydrograph**, a plot of flow rate vs. time that is measured at a stream cross section, is made up primarily of various contributing flows. Base flow

**Figure 1-20**

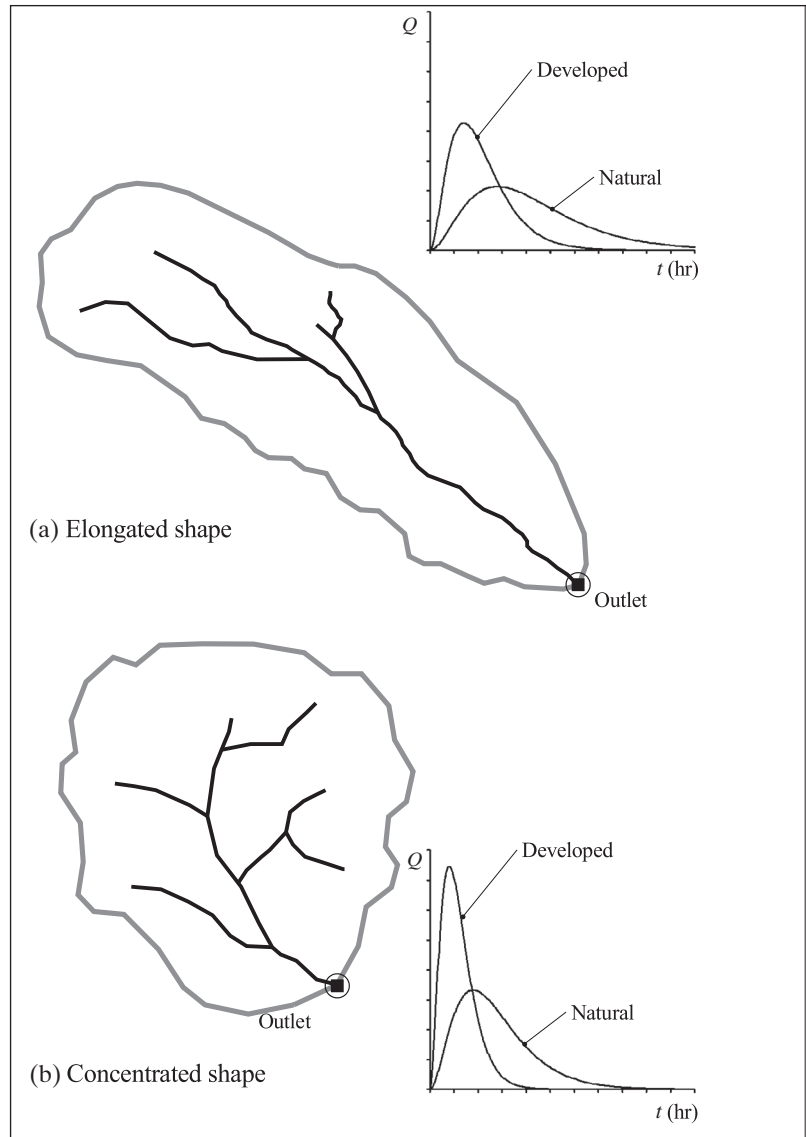
Distribution of uniform rainfall presented with a runoff hydrograph. Note the parameters of infiltration, depression storage, and detention storage.

can also contribute and is produced from soil moisture and ground water contributions. The actual shape and timing of the hydrograph is determined largely by the size, shape, slope, and storage in the basin and by the intensity and duration of input rainfall. Chapter 2 presents these concepts in more detailed examples for ultimate hydrograph analysis. Evaporation and infiltration losses are covered briefly in Section 1.8, and in more detail in Chapter 2.

The classic concept of streamflow generation by **overland flow** is due to Horton (1933), who proposed that overland flow is common and areally widespread. Later investigators incorporated concepts of heterogeneity, which exists across natural watersheds, and developed the partial area contribution concept (Betson, 1964). This model recognized that only certain portions of a watershed regularly contribute overland flow to streams and that no more than about 10% of a watershed contributes overland flow. In urban environments with large areas of impervious cover, the overland flow percentage may be larger. Significant surface runoff occurs only after rainfall intensity exceeds infiltration capacity, or ($i > f$) and soil moisture storage has been filled. Infiltration f is the loss rate into the soil system.

The channel may contain a certain amount of **baseflow** coming from ground water and soil contributions even in the absence of rainfall. Discharge from rainfall excess, after infiltration losses have been subtracted, makes up the **direct-runoff hydrograph (DRO)**. The total storm hydrograph consists of direct runoff plus baseflow. The duration of rainfall determines the portion of watershed area contributing to the peak, and the intensity of rainfall determines the resulting peak flow rate. If rainfall maintains a constant intensity for a very long time, maximum storage is achieved and a state of equilibrium discharge is reached, where the hydrograph tends to level off to a constant value for a period of time. The condition of equilibrium discharge is seldom attained over a large basin in nature because of the variability of actual rainfall.

Figure 1-21 depicts several hydrographs for a given rainfall from watersheds of similar size, but with differing shape and land use characteristics. As

**Figure 1-21**

Land use effects on the hydrograph for both types of watershed shape. For a natural watershed, it can be seen that the elongated shape's hydrograph is longer and has a smaller peak flow than the concentrated shape's. In both cases, however, the effect of development shows a decrease in timing and an increase in peak flow. Development has a greater effect on the concentrated shape.

a watershed develops with urbanization through time, the response normally increases in peak flow and decreases in terms of the time of peak. This is due to increased impervious areas and greater channel density that speeds runoff toward the outlet of the basin (see Chapters 6 and 9). As can be seen, the shape and timing of the hydrograph are largely related to watershed shape characteristics, and it is a central problem of hydrology to understand and provide tools to predict these relationships. As watersheds become more complex as they develop or urbanize through time, it becomes necessary to use hydrologic computer models to simulate watershed response to a given input rainfall and land use pattern (see Chapters 5, 6, and 7). Unit hydrograph analysis and computational methods are discussed in more detail in Chapter 2 along with the effects of land use changes and urbanization, as well as advanced methods for addressing overland flow.

Surface Runoff Phenomena

1.7 HYDROGRAPH ANALYSIS

Figure 1–22 shows a typical watershed area that receives rainfall input. Rainfall that falls on the watershed makes its way from west to east across the area and finally flows out at the outlet. Meteorological factors, physiographic or watershed factors, and human factors (i.e., land use cover) all contribute to the response at the outlet. Note that subareas G and F flow out through area D, then through B (areas E and C then flow into the confluence where B and C flow together), and finally flow through A and contribute to the

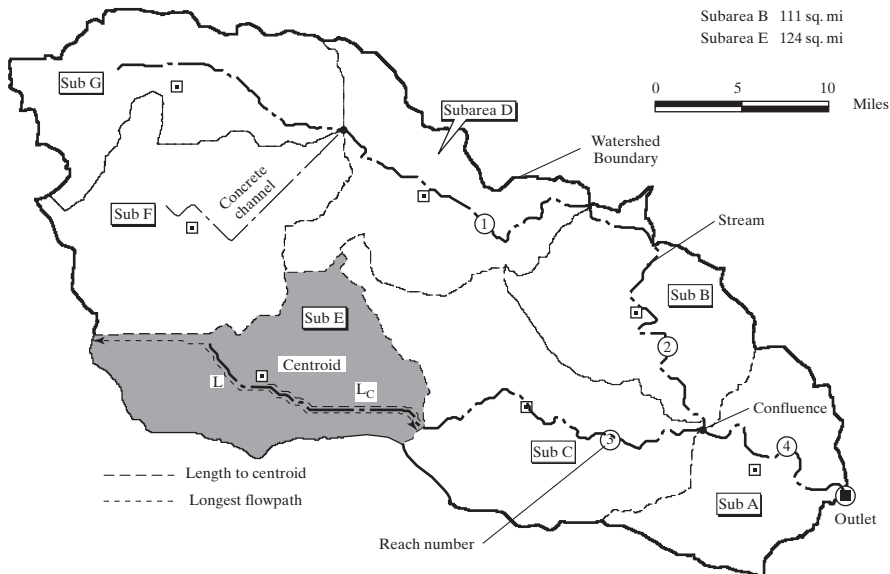
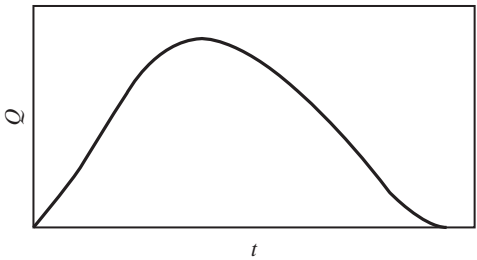
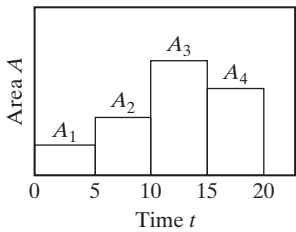
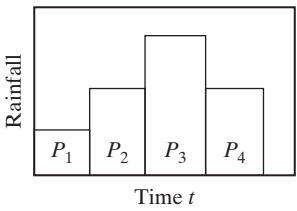
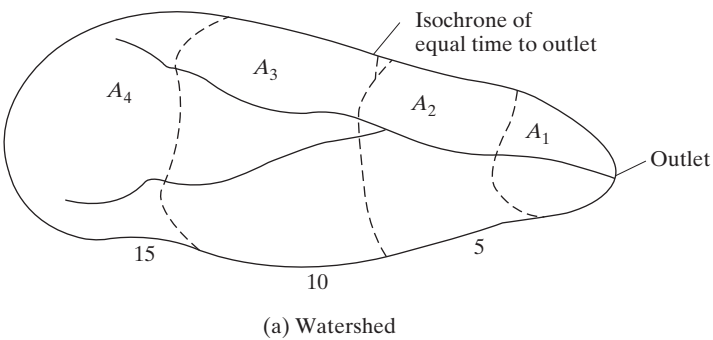


Figure 1–22
Typical subwatershed delineation in an elongated watershed.

outlet. Hydrographs are flood routed through the stream reaches that are numbered 1 to 4 (Chapter 4).

Figure 1–23 illustrates how the **time area histogram** is used to compute the hydrograph response from a watershed. The concept assumes that the hydrograph is built up by various contributions from areas of equal travel time from the outlet. Isochrones define the subareas and travel times and thus a complex rainfall event can be analyzed by computing products of rainfall P_i and area A_i . Surface runoff from A_1 arrives at the outlet first, followed by A_2 , A_3 , and so on. Note that rainfall from period P_1 falling on A_2 arrives at the outlet at the same time as rainfall from period P_2 falling on A_1 , and produces outflow Q_2 . The hydrograph peak occurs when all areas of the watershed are contributing to the outlet.

Figure 1–23
Time Area Method
for a hypothetical
watershed with four
rainfalls recorded
and four areas delin-
eated by isochrone
lines. This method
is used to create a
hydrograph at the
outlet.



The Clark unit hydrograph in HEC-HMS is based on using a time area histogram (Chapter 5).

Often the time duration of rainfall is usually much shorter than the time base of the hydrograph. An actual rainfall (in.) and associated hydrograph (cfs) is depicted vs. time in Figure 1–24 for Little Cypress Creek in Houston for several periods of intense rainfall. It can be seen that the hydrograph rises to a peak flow and then recedes to a zero flow after about 40 hr. The early part of the 3.3 in. rainfall infiltrated into the soil surface, and resulted in about 50% as direct runoff contributing to the hydrograph. The student should verify this by estimating the volume under the hydrograph (3.5 sq mi basin) using a triangle to approximate the area, and compare to the recorded rainfall.

An interesting concept is that of uniform rainfall occurring for an extended time over a small watershed. If rainfall continues at a constant intensity for a very long period, storage is filled at some point, and then an **equilibrium discharge** can be reached such that inflow and outflow are equal (Fig. 1–25). The point *P* indicates the time at which the entire discharge area contributes to the flow, the time of concentration. The condition of equilibrium discharge is seldom observed in nature, except for very small basins or parking lots, because of natural variations in rainfall intensity and duration. This concept is used later when the S-curve is discussed for unit hydrographs in Chapter 2.

Some precipitation may percolate to the water table, usually located 10 to 100 feet below the ground surface in porous media, and may slowly contribute flow to a stream if the water table intersects the stream channel. **Base flow** in a natural channel is due to these contributions from shallow ground water and contributes some flow to a hydrograph. A simple constant

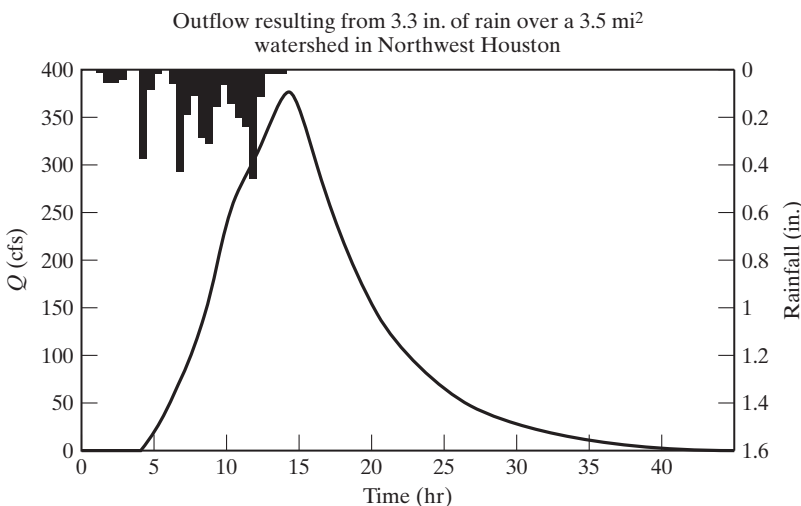
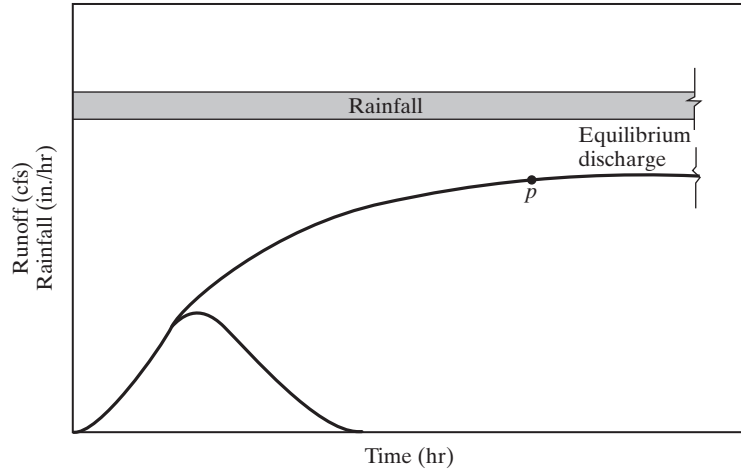


Figure 1–24

Actual rainfall and hydrograph resulting from 3.3 in. of rain for little Cypress Creek.

Figure 1-25

Equilibrium Hydrograph. A result of uniform rainfall over a small watershed in which equilibrium discharge is reached when inflow and outflow are equal.



base flow is shown in Figure 1-26. In large natural watersheds or river basins, base flow may be a significant fraction of streamflow, while it can often be neglected in small, urbanized streams where overland flow predominates. Base flow can be separated from the total storm hydrograph by a number of methods (Fig 1-27), described later, in order to derive the **direct runoff (DRO) hydrograph**.

A typical **hydrograph** is characterized by (1) a **rising limb**, (2) a **crest segment**, and (3) a **recession curve**, as shown in Figure 1-26. The inflection

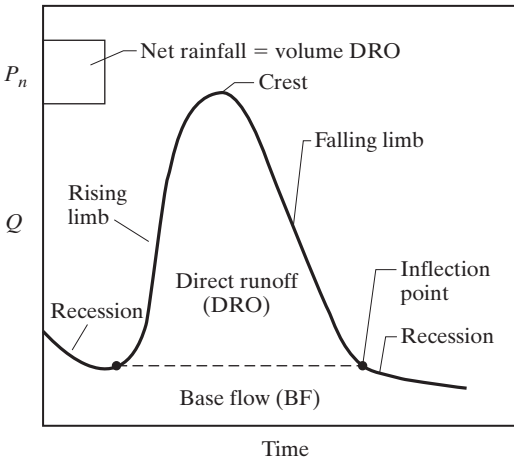


Figure 1-26

Components of an outflow hydrograph. When baseflow is removed, the result is the direct runoff outflow (DRO).

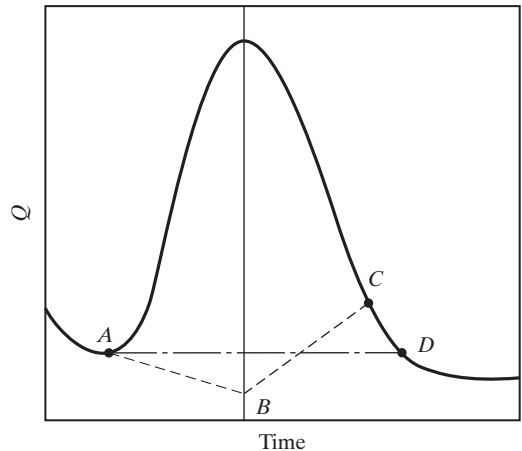


Figure 1-27

Two methods for base flow separation (Straight line and Concave).

point on the falling limb is often assumed to be the point where direct runoff ends. Rainfall excess P_n is obtained by subtracting infiltration losses from total storm rainfall (Fig. 1–26), while evaporation can usually be neglected for an individual storm event on a small basin. The DRO represents the hydrograph response of the watershed to rainfall excess P_n , defined as gross precipitation minus infiltration. The shape and timing of the DRO hydrograph are related to the duration and intensity of rainfall as well as the various factors governing the watershed area.

Infiltration and the Hydrograph Response

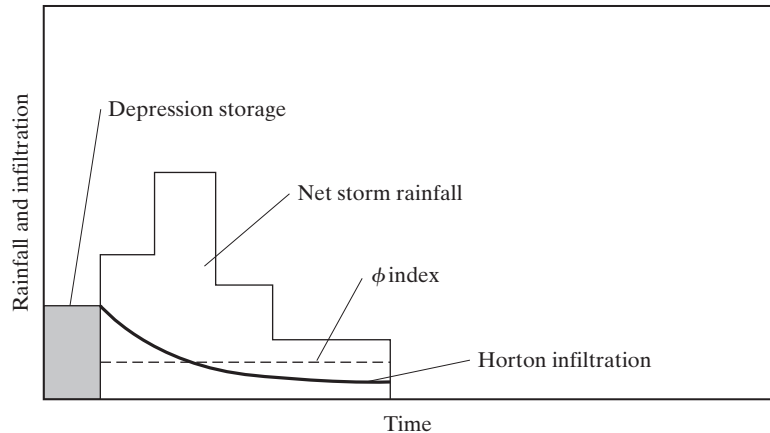
The relative contribution of each component to the hydrograph is dependent on rainfall rate i relative to the infiltration rate f of the soil. Infiltration loss concepts are presented in detail in Sections 2.7 and 2.8, where several methods are presented that can handle most cases of variable infiltration and variable rainfall intensity. The contributions also depend on the level of **soil moisture storage** S_D vs. **field capacity** F of the soil, which is defined as the amount of water held in place after excess gravitational water has drained.

No overland runoff occurs for the case $i < f$, since all rainfall infiltrates, and interflow and ground water flow are zero if $F < S_D$, since soil moisture storage exists and can handle additional inflows. In this first case, no measurable hydrograph is produced, and all rainfall infiltrates as in the case of a small rainfall under very dry soil conditions.

However, the extreme case where both $i > f$ and $F > S_D$ is typical of a large storm event in which direct surface runoff, interflow, and base flow all contribute to the hydrograph. In this case, the intensity of rainfall dominates the system, and large quantities of overland flow and channel flow are produced. Channel precipitation is usually a very small fraction of total flow rate and is usually neglected in practice. In urban areas, the infiltration component may be minor where concrete surfaces and channels dominate the response. In more forested and natural watersheds, infiltration and surface roughness may be major factors and significantly alter the hydrograph.

Infiltration is one of the most complex of hydrologic phenomena. Horton (1933) showed that when the rainfall rate i exceeds the infiltration rate f , water infiltrates the surface soils at a rate that generally decreases with time (Fig. 1–28). These rates are usually reported in inches per hour. For any given soil, a limiting curve defines the maximum possible rates of infiltration vs. time. The rate of infiltration depends in a complex way on rainfall intensity, soil type, surface condition, and vegetal cover. Sand will generally infiltrate at much greater rates than silt or clay. Infiltration from a large basin can represent as much as 25% to 30% of rainfall, but usually is in the 5% to 15% range.

The **ϕ index** is the simplest infiltration method and is calculated by finding the loss difference between gross precipitation and observed surface runoff measured as a hydrograph. The ϕ index method assumes that the loss

**Figure 1-28**

Infiltration loss curves used to find rainfall excess. Horton's infiltration curve and the ϕ index with initial loss for depression storage.

is uniformly distributed across the rainfall pattern (Fig. 1-28). Sometimes the method is modified to include a greater initial loss or abstraction followed by a constant loss for the event. The use of ϕ index methods for infiltration is illustrated in Example 2-11.

Recession and Base Flow Separation

It is generally customary to consider the hydrograph to be divided into only two parts: direct runoff (DRO) and base flow (BF), as shown in Figures 1-26 and 1-27. DRO may include some interflow, whereas base flow is considered to be mostly from contributing ground water. In most urban streams, base flow will be a relatively small component, usually less than a few percent, while it can be a significant component in a large river basin.

Several techniques exist to separate DRO from base flow based on the analysis of ground water recession curves. In many cases, the recession curve can be described by an exponential depletion equation of the general form

$$q_t = q_0 e^{-kt}, \quad (1-14)$$

where

- q_0 = specified initial discharge,
- q_t = discharge at a later time t ,
- k = recession constant.

Equations of this form are used often in engineering to describe first-order decay or depletion. Equation (1-14) will plot as a straight-line depletion curve on semilogarithmic paper, and the difference between this curve and

the total hydrograph plotted on the same paper represents the DRO. Several storms should be used in order to develop a master depletion curve (McCuen, 2005). In practice, there are three simpler methods for handling base flow separation. The base flow recession can be extended forward under the peak of the hydrograph, starting with the point of lowest discharge and then extending at constant discharge to a point on the recession limb (AD in Fig. 1–27). The inflection point on the recession is assumed to be the point where the DRO ends. A second method is the concave method, where base flow is extended under the peak of the hydrograph and then is connected to the inflection point on the recession curve (ABC). Figure 1–27 shows two of the above methods. Computerized methods for base flow separation are also available (Sloto and Crouse, 1996).

The separation methods all suffer the disadvantage of being arbitrary and somewhat inaccurate. At present, base flow separation is more an art than a science. In many cases of practical interest such as urban drainage, base flow is often neglected because it represents such a small component. Base flow is usually more important in natural streams and large rivers because of the contribution along the banks from a water table.

Net Storm Rainfall and the Hydrograph

Surface runoff phenomena have been discussed above, and the distribution of gross rainfall can be divided into components of infiltration, depression storage, detention storage, and direct runoff. We can write the continuity equation for this process as

$$\begin{aligned} \text{gross rainfall} = & \text{depression storage} + \text{evaporation} + \text{infiltration} \\ & + \text{surface runoff}, \end{aligned}$$

where detention storage is included as eventual surface runoff. In cases where depression storage is small and evaporation can be neglected, one can compute rainfall excess, which equals direct runoff, DRO, by

$$\begin{aligned} \text{rainfall excess } (P_n) = & \text{DRO} \\ = & \text{gross rainfall} - (\text{infiltration} + \text{depression storage}). \end{aligned}$$

It is often important to determine the time distribution for rainfall excess, P_n . Generally, the simpler methods employed to determine rainfall excess include the Horton infiltration method and the ϕ index method, with initial loss included for depression storage, as shown in Figure 1–28. Loss volumes can be determined by comparing total rainfall and hydrograph volumes using various infiltration equations (i.e., compute in./hr over the watershed area), as described in detail in Example 1.6. The Horton method estimates infiltration with an exponential-type equation that slowly declines in time as rainfall continues; the ϕ index infiltration methods is based on a constant rate in in./hr through the rainfall period. In practice, the loss-rate coefficients are difficult to estimate, and the simpler ϕ index method is often

used, probably because of the lack of data on infiltration distribution in time. Note that the ϕ index tends to underestimate losses at the beginning of the storm and overestimate losses at the end. More advanced infiltration methods (Green and Ampt, 1911; Mein and Larson, 1973) can also be employed where detailed soils data and hydraulic conditions are available, as described in Chapter 2.

EXAMPLE 1-6

NET STORM RAINFALL

Rain falls as shown in the rainfall hyetograph (intensity vs. time) in Figure E1-6a. The ϕ index for the storm is 0.5 in./hr and is constant over 5 hr. Plot the net rainfall on the hydrograph (flow vs. time) given in Figure E1-6a. Determine the total volume of runoff and the watershed area.

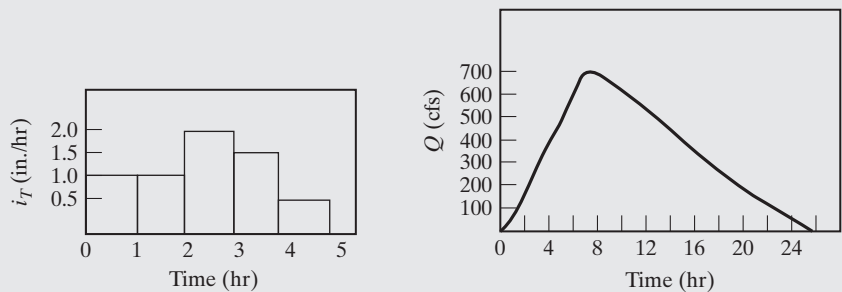


Figure E1-6a
Rainfall hyetograph and hydrograph.

SOLUTION

First we develop the net rainfall hyetograph shown in Figure E1-6b. Then, we add this to the hydrograph plot in the upper left corner of Figure E1-6c. Note that the rainfall excess becomes 3.5 inches total with a duration of 4 hr, since rainfall equals infiltration during the final hour of rainfall.

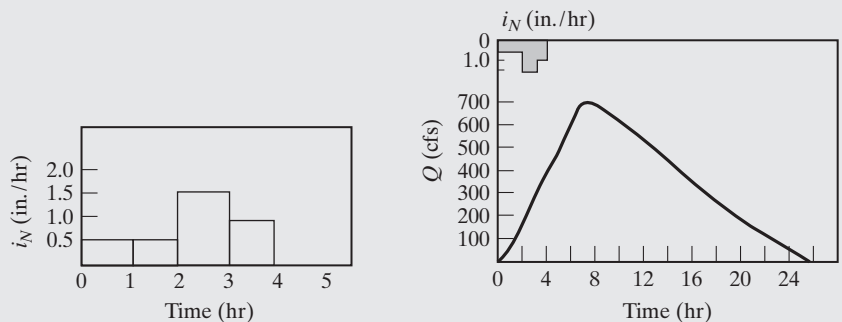


Figure E1-6b
Net rainfall hyetograph.

Figure E1-6c
Final hydrograph plotted with net rainfall.

The volume of runoff is equal to the area under the hydrograph. To determine the volume of runoff, we can use $\Sigma \bar{Q} dt$. This estimates the volume as the bar graph shown in Figure E1-6d. Calculations are tabulated in the accompanying table.

Time (hr)	$\bar{Q}$ (cfs)	Volume (cfs-hr)
0-2	100	200
2-4	300	600
4-6	500	1000
6-8	700	1400
8-10	650	1300
10-12	600	1200
12-14	500	1000
14-16	400	800
16-18	300	600
18-20	200	400
20-22	150	300
22-24	100	200
24-26	50	100

$$\Sigma \bar{Q} dt = 9100 \text{ cfs-hr}$$

$$= 9028 \text{ ac-in of direct runoff (1 ac-in/hr} = 1.008 \text{ cfs-hr)}$$

$$= 3.47 \text{ inches over the watershed area of 2600 acres.}$$

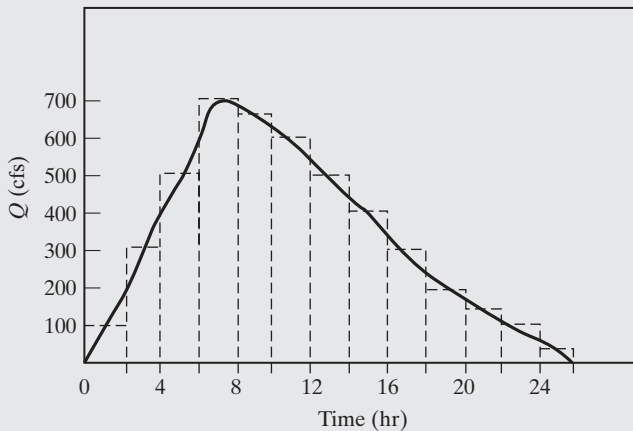


Figure E1-6d

Estimation of runoff volume $\Sigma \bar{Q} dt$ resulting bar graph.

Other methods that could be used with Figure E1–6d include mathematical methods of integration, such as the trapezoidal rule and Simpson’s rule. Both methods approximate the area under the graph hydrograph between two values of time. If base flow were included in the hydrograph, we would have to subtract it first from the flow values at each time step before computing volumes of direct runoff.

Once rainfall excess has been determined for a watershed, it then becomes a central problem of engineering hydrology to convert it into direct runoff, DRO. The resulting hydrograph is basically built up from contributions of overland flow and channel flow arriving at different times from all points in the watershed. The relative times of travel of overland and channel flow are related to the size of the watershed; overland flow time is more significant in a small watershed, whereas time of travel in the channel predominates in a large watershed.

A number of investigators have attempted to develop rainfall–runoff relationships that could apply to any region or watershed under any set of conditions. The Soil Conservation Service (SCS, 1964, 1986) presented a useful set of rainfall–runoff curves that also include land cover, soil type, and initial losses (abstraction) in determining direct runoff (Section 2.3). A number of unit hydrograph methods are presented in Chapter 2, and Snyder’s method, the TC & R method, and the SCS methods are most often used based on their simplicity and relative accuracy under a variety of watershed conditions.

1.8 HYDROLOGIC MEASUREMENT

Hydrologic processes vary in space and time, and accurate measurement requires the use of sophisticated instruments, which are usually at a fixed location in space. The resulting data from rainfall, evaporation, or stream-flow levels measured over time create a time series, which may be further analyzed statistically. The overall sequence of steps for hydrologic measurement includes

1. Sensing, which transforms the intensity of the process into a measurable signal;
2. Recording, usually in electronic form;
3. Transmitting to a central processing site via telemetry;
4. Translating, which converts the record into a data sequence;
5. Editing, or checking for any errors in the data;
6. Storing, in a computer database for viewing;
7. Retrieving the data for further use.

The advent of advanced telemetry, radar systems, use of the Internet, and use of large database storage devices has revolutionized hydrologic information storage, retrieval, and distribution. A number of useful websites often used in hydrology for rainfall and streamflow data are listed in Appendix E.

Atmospheric Parameters and Precipitation

The measurement of atmospheric moisture near the ground often utilizes a climate station, which usually consists of a psychrometer (or hygrometer) to measure humidity, a rainfall gage, an evaporation pan, and an anemometer for wind speed and direction. Total incoming or outgoing radiation can be measured with a radiometer. Weather balloons are used to measure temperature, humidity, air pressure, and wind speed at various elevations above the earth.

The fundamental instrument for measuring atmospheric pressure is the mercurial **barometer**, which is constructed by filling a long glass tube with mercury. The barometer acts as a weighing balance, and changes in atmospheric pressure are detected from changes in the height of the column of mercury. The **psychrometer** is an instrument based on temperature differences between two thermometers, one of which is covered in a wet cloth, called the wet bulb. The difference in temperature of the dry and wet bulbs, when ventilated, is a measure of degree of saturation of the air, which is a measure of relative humidity.

Precipitation or rainfall is measured at a point with either non-recording or recording gages. The non-recording gage consists of a simple open tube with readings along the side for visually determining rainfall amounts. Recording gages are required in urban settings for flood control studies if the time distribution of rainfall is desired. The tipping bucket recording gage operates off a pair of small buckets (Fig. 1-14a), which move in a flip-flop motion and make electrical contact for every increment of rainfall. The buckets direct water into a reservoir, which can be checked for cumulative totals at the end of a storm event. The gage usually records every 0.01 in. or 0.1 in. of rain and telemeters this to a central computer for storage and analysis.

Radar (radio detection and ranging) has been used to measure rainfall rates on the ground since 1994, and GIS methods have greatly improved our ability to determine rainfall rates for specific watershed areas. Chapter 11 presents more details.

Evaporation and Infiltration Losses

Evaporation is the process by which water in its liquid or solid state is transformed into water vapor, which mixes with the atmosphere. **Evapotranspiration** (*ET*) is considered separately as the combined loss of water vapor from the surface of plants (transpiration) and the evaporation of moisture from soil. Knowledge of evaporation processes is important in predicting water losses to evaporation from a lake or reservoir. On average, approximately 70% of the mean annual rainfall in the United States is returned to the atmosphere as evaporation or transpiration. However, variations in evaporation across the continent can be very large due to effects of solar input, location of mountains, and proximity to oceans. On an annual basis, evaporation rates can exceed mean annual rainfall, especially in arid regions

of the southwest. For example, arid areas in Texas, Arizona, New Mexico, Nevada, and southern California can exceed 70 inches per year, compared to about 30 to 40 inches per year for much of the rest of the country. For the case of evaporation from a lake surface, water loss is a function of solar radiation, temperature of the water and air, difference in vapor pressure between water and the overlying air, and wind speed across the lake.

Evaporation is important in the long-term water balance and is usually of concern for large-scale water resources planning and water supply studies, especially in the Western states where evaporation rates can exceed rainfall rates. During typical storm periods, with intensities of 0.5 in./hr, evaporation is on the order of 0.01 in./hr and is normally neglected for flood flow studies and urban drainage design applications.

Measurement of evaporation is usually from a standard class A pan (Fig. 1-29), which is filled to 8 in. and then observed on a daily basis. Adjustments are made for rainfall input using a rainfall gage nearby. Methods to compute and measure evaporation and evapotranspiration are covered in detail in Section 2.6. Evaporation and ET are very difficult to measure accurately for large watersheds.

Infiltration, or movement of water from the surface into the soil zone, can be measured with a ring infiltrometer, which is a ring about 2 ft in diameter driven into the soil. Water is placed in the ring, and the rate of infiltration, f , is measured by the drop in water level over time. Carefully controlled experiments require the use of tensiometers to measure capillary suction with depth, electrical resistance to record moisture content, and wells to record the response at the water table below. Infiltration is one of the most difficult hydrologic parameters to measure because of the extreme variations in soil and water conditions, which can greatly affect the measured rate. In actual watersheds, infiltration is often determined by the difference between gross rainfall and direct runoff measured from a hydrograph. Both simple and advanced infiltration methods are presented in Sections 2.7 and 2.8.

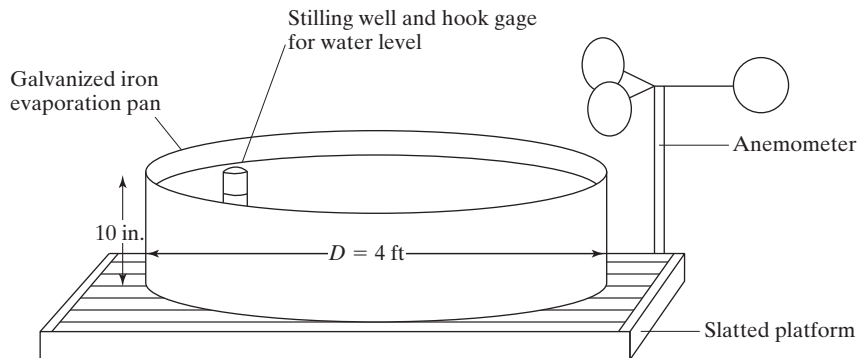


Figure 1-29

Standard class A evaporation pan with cup anemometer and rain gage.

Streamflow Measurement

Streamflow is generally measured by observing stage, or elevation above a specified elevation datum (i.e., mean sea level), in a channel and then relating stage to discharge via a rating curve. A staff gage is a fixed scale set so that a portion is immersed in water and can be read manually during storm passage. A wire-weight gage is lowered from a bridge structure to the water surface, and readings are taken as a function of time through a storm event. A crest stage gage uses a small amount of cork inside an enclosed staff gage. The cork floats as the water rises and adheres to the scale recording at the highest water level.

Most automatic recording gages, such as those used by the USGS for routine streamflow monitoring, use a float-type device to measure stage or a gas bubbler to measure pressure (Fig. 1–30a). The bubble gage senses water level by maintaining a continuous stream of gas in a small pipe under the water. Another approach is to use a pressure transducer near the bottom of a stream that senses the pressure or height of water that sits above the monitor. In some urban basins, hydrologic data are often transmitted over telephone lines or through telemetry directly into computers. Acoustic Doppler flow meters (ADFM) have recently emerged as a new method to accurately measure flow rate by measuring the actual velocity profile in a pipe or open channel.

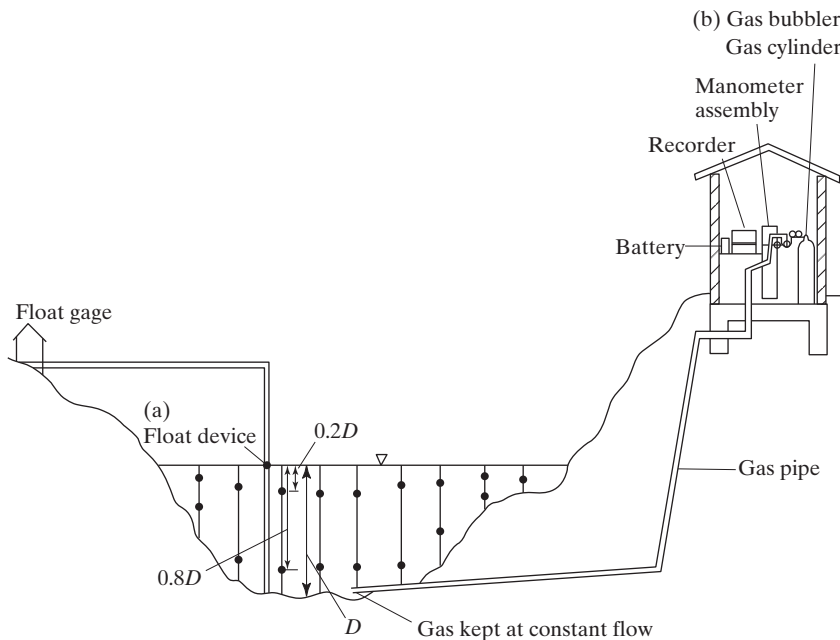
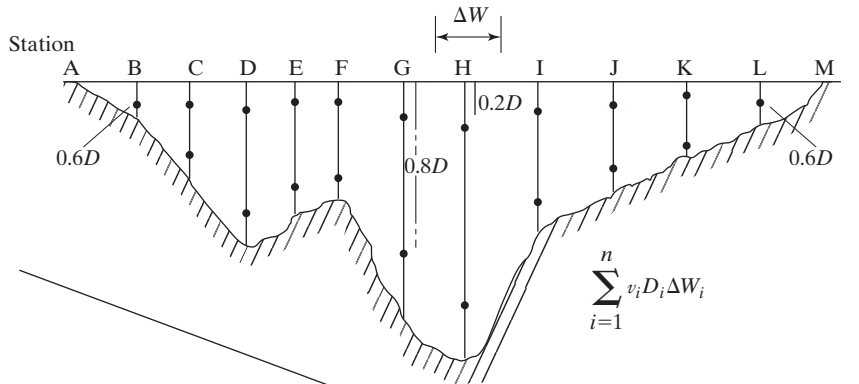


Figure 1–30a

Typical USGS streamflow gages with a float-type device to measure stage (a) or a gas bubbler to measure pressure (b).

**Figure 1-30b**

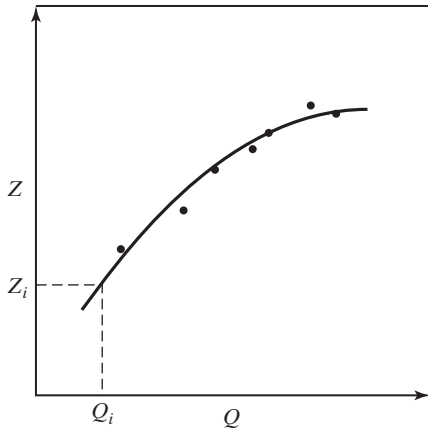
Stream section divided to find total discharge. Each section area is multiplied by the average velocity within that section and then summed up to yield total discharge.

Selection of a site for stream gaging must include consideration of access, channel controls where flow rate and depth are related, and seasonal changes in vegetation. Most USGS sites have been carefully selected to incorporate access factors.

Once a stream station has been established, usually at a bridge crossing, a rating curve can be developed between stage and discharge by actually measuring velocity in the channel at a number of different stages with a current meter. The current meter is suspended from a bridge or held by a rod in shallow water and records velocity according to the rotational speed of the propeller. The recommended procedure for determining mean velocity is to take measures at $0.2D$ and $0.8D$ (down from the surface), where D is stream depth, and average the two values (see Fig. 1-30b).

The total discharge is found by dividing the channel into several sections, as shown in Figure 1-30b. The average velocity of each section is multiplied by its associated area (width times depth of section), and these are summed across the channel to yield total discharge Q corresponding to a particular stage z , obtaining one point on the graph in Figure 1-31. Other points are obtained by measuring velocity at different stages in the stream. Rating curves can change through time, as watersheds change in terms of land use and channel type, and should be rechecked periodically.

It will be shown in Chapter 4 that actual rating curves can be looped because of storage and hydraulic effects in the channel, and the single-valued rating curve is at best an approximation to the actual relationship (see Fig. 4-14). A more detailed discussion of streamflow prediction and measurement is contained in Chapter 4.

**Figure 1-31**

Rating curve. A rating curve is obtained for a particular cross section by finding the total Q at a particular stage z . The other points are obtained by finding different velocities to obtain Q at different stages. These can change as watersheds change due to land use and channel types.

Conventional stream gaging depends on a collection of stage measurements and occasional current-meter discharge measurements upstream of an open-channel control in a stream or river. The USGS has more than 10,000 conventional gaging systems in operation, and its website is listed in Appendix E. However, for urban gaging situations, where the range in stage is not excessive, specialized gaging systems have been developed. Figure 1-32 shows a typical urban hydrology gaging system being used in a concrete-lined urban stream in Texas. This type of stream is often composed of a stage recording mechanism (bubbler type), a recording rain gage, a crest stage gage indicator, and two independent staff gages. The gaging system is equipped with two automatic digital recorders, one for stage and one for rainfall, with data recorded and stored at approximately 5-min intervals. The station is equipped with telemetry for radio signals back to a base station or computer.

**Figure 1-32**

U.S. Geological Survey stream gage station with telemetry.

Flood Alert Systems

In recent years, with the advent of software for personal computers and advanced electronic transmitters, hydrologists have designed flood alert systems to collect, transmit, and analyze data from remote gages in a large watershed. The Harris County Office of Emergency Management (HCOEM) near Houston, Texas, has implemented such a system from Sierra Misco, Inc. (1986), and many others are in existence around the United States. The remote stations can provide radio signals for both rainfall and streamflow gaged data on a real-time basis during the passage of a large storm event. The data are sent directly to a base station, where the analog signals are converted to digital for computer storage and analysis. If the incoming data are from a remote location, repeater antennae are used to intercept the remote signal and send it to the base station.

Figure 1–33 shows data flow through a typical flood alert system on a real-time basis. The Texas Medical Center in Houston has implemented

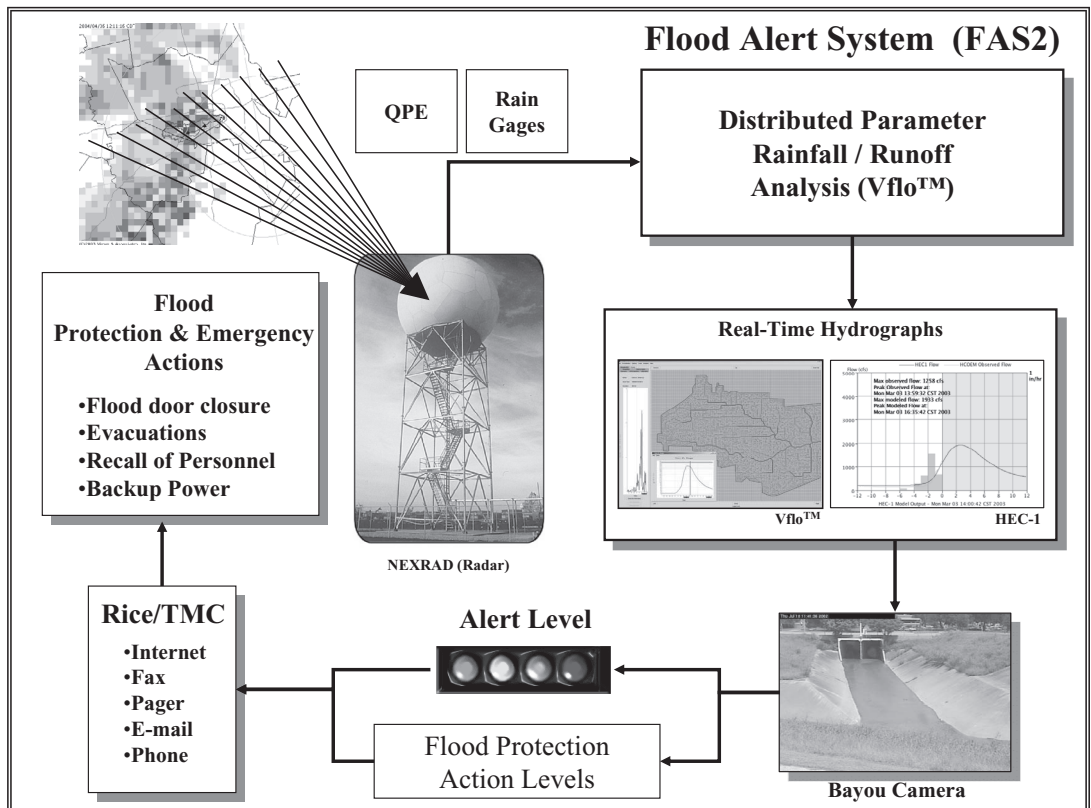


Figure 1–33
Typical RADAR-based flood alert system.

a unique flood alert system (FAS3) to provide flood information to the over 22 hospitals located in the Brays Bayou watershed (Bedient et al., 2003, 2007). This system allows emergency vehicles and personnel to better plan for the unexpected during a large storm event over the watershed. The FAS2 relies on NEXRAD data to estimate rainfall intensities and cumulative amounts over a 100-sq-mi watershed in southwest Houston. More details on FAS3 are provided in Chapters 11 and 12, where NEXRAD radar-rainfall and hydrologic models are presented in more detail in Fang et al., 2011.

Chapter 1 has covered the basic principles of hydrology, including the water balance, weather systems, precipitation, streamflow, simple hydrograph analysis, hydrologic losses, and hydrologic measurement. Solar radiation and atmospheric water phase changes provide the main energy inputs, and result in the generation of precipitation. Once rain falls to the earth, it can infiltrate into the soil system, percolate to deeper ground water, evaporate back to the atmosphere, or generate runoff to the nearest stream or river. Overall water balance is maintained through the various storage mechanisms within the hydrologic cycle, and detailed examples are provided. The ocean is the ultimate receptor of surface flow from rivers and channels, and provides the main source of water for evaporation back to the atmosphere.

SUMMARY

The atmosphere is the major hydrologic link between oceans and continents on the planet, facilitating the cycle of movement of water on earth. The hydrologic cycle is largely shaped by the conditions of the atmosphere, with precipitation as the main input when the atmospheric conditions are unstable. Solar radiation, general circulation, wind systems, moisture source, water vapor content, and lifting mechanisms are all covered in the chapter. As moist air rises, phase change from vapor to liquid with release of latent heat is an important driver supplying atmospheric energy. Major thunderstorms and hurricanes are the result of unstable atmospheres and strong vertical movements, and produce major rainfall and damaging winds and tornadoes.

The concept of the watershed, a basin area that drains to a single outlet, as the basic hydrologic unit is defined with examples. Watershed response from a given rainfall depends on size, shape, slope, soils, storage, and land use within the area. The actual response is often plotted as a hydrograph of flow rate Q vs. time t . Precipitation input is the main driver of the hydrologic cycle, as it relates to river flow, water supply, flooding, agricultural and urban drainage, and recharge to ground water.

Simple hydrograph analysis, the conversion of rainfall into runoff overland or in a stream, is covered in Chapter 1, and expanded with the unit hydrograph in Chapter 2. Point, areal, design, and radar-based rainfalls are described in some detail as input to the creation of the hydrograph from a watershed area. Precipitation, from gages or radar, and streamflow can be directly measured by methods detailed in Section 1.8.

**CONCEPT
CHECK**

- 1.1. What is the hydrologic cycle? What are the pathways that precipitation falling onto the land surface of the earth is dispersed to the hydrologic cycle?
- 1.2. Who is responsible for the first recorded rainfall measurements? Describe the technique used to obtain these measurements.
- 1.3. Explain the difference between humidity and relative humidity.
- 1.4. Explain how air masses are classified. Where are these types of air masses located?
- 1.5. List seven major factors that determine a watershed's response to a given rainfall.

PROBLEMS

- 1.6. A lake with a surface area of 750 acres was monitored over a period of time. During a one-month period, the inflow was 35 cfs, the outflow was 28 cfs, and a 1.5-in. seepage loss was measured. During the same month, the total precipitation was 4.8 in. Evaporation loss was estimated as 6.0 in. Estimate the storage change for this lake during the month.
- 1.7. Clear Lake has a surface area of 610,000 m² (61.0 ha). For a given month, the lake has an inflow of 1.5 m³/s and an outflow of 1.25 m³/s. A +1.0-m storage change or increase in lake level was recorded. If a precipitation gage recorded a total of 20 cm for this month, determine the evaporation loss (in cm) for the lake. Assume that seepage loss is negligible.
- 1.8. In a given month, a watershed with an area of 1500 m² received 100 cm of precipitation. During the same month, the loss due to evaporation was 15 cm. Ignore losses due to transpiration and infiltration due to ground water. What would be the average rate of flow measured in a gage at the outlet of the watershed in m³/day?
- 1.9. In a given year, a watershed with an area of 2000 km² received 120 cm of precipitation. The average rate of flow measured in a gage at the outlet of the watershed was 30 m³/sec. Estimate the water losses due to the combined effects of evaporation, transpiration, and infiltration due to ground water. How much runoff reached the river for the year (in cm)?
- 1.10. Using the data from problem 1.9, what is the runoff coefficient?
- 1.11. Plot Equation (1-4) as a graph (e_s vs. T) for a range of temperatures from -30°C to 40°C and a range of pressures from 0 to 70 mb. The area below the curve represents the unsaturated air condition. Using this graph, answer the following:
 - (a) Select two saturated and two unsaturated samples of air from the dataset of pressure and temperature given below:
 Pressure (mb): {10, 20, 30}
 Temperature (°C): {10, 20, 30}

- (b) Let A and B be two air samples, where A: ($T = 30^\circ\text{C}$, $P = 20$ mb) and B: ($T = 30^\circ\text{C}$, $P = 40$ mb). For each sample, determine the following:
- Saturation vapor pressure
 - Dew point
 - Relative humidity
- (c) Suppose both samples A and B were cooled to 20°C . What would be their relative humidity? What would be their dew point temperature?

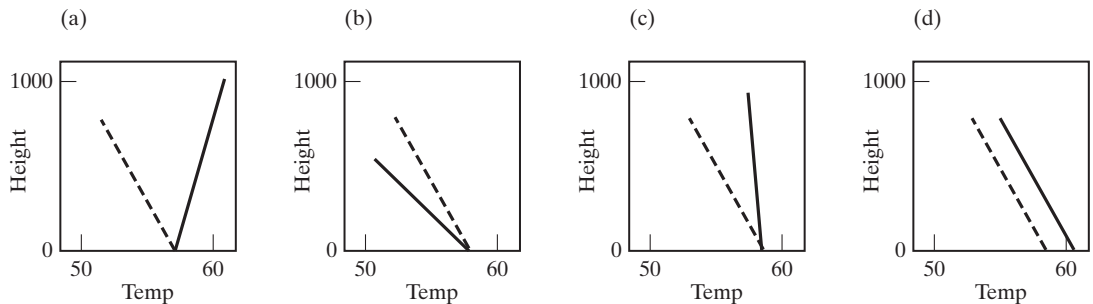
1.12. The gas constant R has the value $2.87 \times 10^6 \text{ cm}^2/\text{s}^2 \text{ }^\circ\text{K}$ for dry air, when pressure is in mb. Using the ideal gas law ($P = \rho RT$), find the density of dry air at 25°C with a pressure of 1150 mb. Find the density of moist air at the same pressure and temperature if the relative humidity is 65%.

1.13. At a weather station, the air pressure was measured to be 95.1 kPa, the air temperature was 22°C , and the dew point temperature was 18°C . Calculate the corresponding vapor pressure, relative humidity, specific humidity, and air density. First compute e and e_s .

1.14. What are the three main mechanisms for generation of vertical air motion?

1.15. Describe the naming system for describing basic clouds.

1.16. Below are three different atmospheric systems. The dashed line represents the dry adiabatic lapse rate whereas the solid line represents the dry ambient lapse rate. Label whether the atmosphere is stable, unstable, or neutral:



1.17. Hurricane Katrina was the most devastating hurricane in U.S. history. Refer to <http://www.hpc.ncep.noaa.gov/>. Find the information concerning total rainfall during Hurricane Katrina, and answer the following questions:

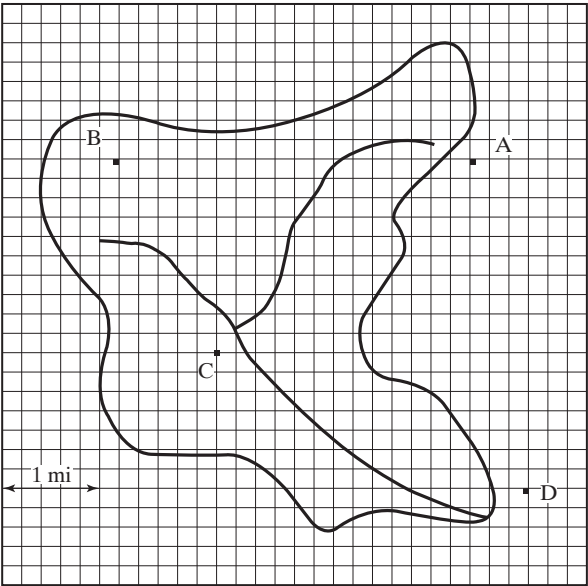
- What category event was Hurricane Katrina when it made landfall?
- Which state experienced the most cumulative rainfall during Hurricane Katrina?
- What was the lowest central pressure recorded during the hurricane?
- What finally halted the path of the hurricane?

1.18. A watershed of 465 ac with six rainfall gages can be divided into Thiessen polygons with the listed data in the accompanying table. Using the total storm rainfall depths listed, find the average rainfall over the watershed in ac-ft.

Gage	Rainfall (in.)	Area (ac)
A	2.20	90
B	3.22	77
C	0.71	17
D	2.49	89
E	0.88	162
F	6.72	30

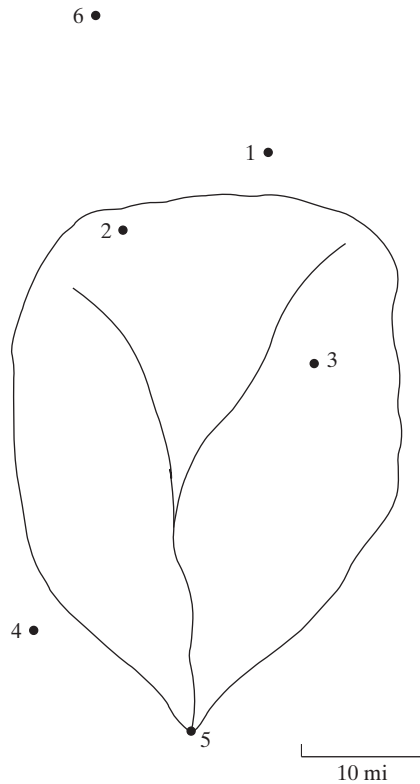
1.19. A small urban watershed has four rainfall gages as located in Figure P1–19. Total rainfall recorded at each gage during a storm event is listed in the table below. Compute the mean areal rainfall for this storm using (a) arithmetic averaging and (b) the Thiessen method.

Figure P1–19
Four rainfall gages
with watershed
boundary.



Gage	Rainfall (in.)
A	3.26
B	2.92
C	3.01
D	3.05

1.20. Mud Creek has the watershed boundaries shown in Figure P1–20. There are six rain gages in and near the watershed, and the amount of rainfall at each one during a storm is given in the accompanying table. Using the Thiessen method and a scale of 1 in. = 10 mi, determine the mean rainfall of the given storm.

**Figure P1–20**

Six rainfall gages
with watershed
boundary.

Gage Number	Rainfall (cm)
1	5.5
2	4.5
3	4.0
4	6.2
5	7.0
6	2.1

- 1.21.** Table P1–21 lists rainfall data recorded at a USGS gage for the storm of September 1, 1999. The basin area is 2050 acres. Using these data, develop a rainfall hyetograph (in./hr vs. t) in 15-min intervals and determine the time period with the highest intensity rainfall.

Problems 1.22 and 1.23 refer to the hydrologic data used in Problem 1.21.

- 1.22.** (a) Plot the cumulative mass curve for rainfall and the hydrograph (flow rate vs. time) on the same graph using two scales.
- (b) Compute the volume of infiltration loss for the storm, neglecting ET by subtracting the volume of gross rainfall from the volume under the hydrograph.

Table P1-21

Time (hr)	Accumulated Rainfall (in.)	Discharge (cfs)
16:05	0	0
16:20	0.4	5
16:35	1.4	600
16:50	1.9	1110
17:05	2.3	1300
17:20	2.5	1380
17:35	2.6	1350
17:50		1280
18:05		1110
18:20		790
18:35		460
18:50		240
19:05		110
19:20		59
19:35		35
19:50		20
20:05		12
20:20		7
20:35		4
20:50		2

- 1.23.** What is the runoff coefficient for the September 1, 1999, storm, where $\text{Coeff.} = R/P$?
- 1.24.** The following questions refer to Figure 1-15, the IDF curve for Houston, Texas.
- What is the return period of a storm that recorded 3.1 in./hr for 2 hr in Houston?
 - What amount of rain (in.) would have to fall in a 6-hr period to be considered a 100-yr storm in Houston?
 - What is the return period of a storm that lasts 1 hr and records 3.5 in. of rainfall?
 - Develop and plot a 6-hr, 100-yr storm design rainfall using 1-hr time steps (see Fig. 1-8). Assume the maximum hourly value occurs between hours 3 and 4.
 - Find the rainfall intensity for a 1-hr duration and plot the rainfall intensity in in./hr between the hours of 3 and 4 on a bar graph.
 - Then, find the rainfall intensity for a 2-hr duration; multiply the intensity by the duration of rain to get the volume. Plot the difference between the 2-hr duration volume and the 1-hr duration volume for hour 2 to 3 on a bar graph.
 - Continue in the same way for the 3-hr duration, plotting the new intensity to the right of the maximum (hour between 4-5). (Find the 3-hr volume and subtract the 2-hr volume.)

- (iv) Then, find the rainfall intensity for a 6-hr duration and the respective volume. Plot the remaining volume (6-hr minus 3-hr) over the 3 hr, assuming equal distribution between them, with two bars to the left and one to the right of the maximum (time intervals 0–1, 1–2, and 5–6).

1.25. Given the stream section shown in Figure P1–25 and the following measurements, calculate the total discharge throughout the section (Table P1–25).

Table P1–25

Measurement Station	Distance Across Stream (ft)	Depth D (ft)	Mean Velocity v (ft/sec)
A	0	0	0
B	14	1.1	0.43
C	26	2.6	0.61
D	38	3.5	1.54
E	49	3.2	1.21
F	61	3.1	1.13
G	78	3.9	1.52
H	95	4.2	2.34
I	114	3.3	1.42
J	133	2.9	1.34
K	152	2.1	1.23
L	171	1.4	0.53
M	190	0	0

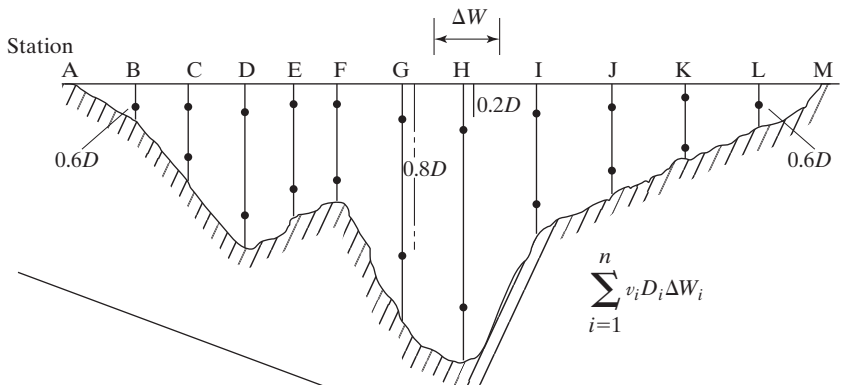


Figure P1–25
Divided stream section.

- 1.26.** Assume that stations B through L in Problem 1–25 have all become 0.2 ft deeper. In addition, a tributary has joined the stream and added approximately 600 cfs to the flow in the channel. Calculate the new discharge amounts for each station by altering the depths and adding the tributary's contribution across the channel in proportion to the modified discharge distribution. Assume velocity distribution remains unchanged.

1.27. The incremental rainfall data in the table were recorded at a rainfall gage on a small urban parking lot of 1 acre. Be careful to use a 0.5-hr time step and record intensity in cm/hr.

- Plot the rainfall hyetograph.
- Determine the total storm rainfall depth (inches).
- If 100% of the rainfall occurs as runoff and the time base of a triangular hydrograph (flow rate vs. time) is 3 hr, find the peak flow of the hydrograph in cfs. Be careful with units.

Hint: First find the volume of rainfall, then equate it to the area under the hydrograph.

Time (hr)	Rainfall (cm)
0	0
0.5	5.0
1.0	5.5
1.5	10.0
2.0	12.0
2.5	10.0
3.0	4.5
3.5	3.0
4.0	2.5
4.5	0

1.28. For the rainfall record provided below, plot cumulative rainfall (P) and gross rainfall hyetograph (in./hr) using $\Delta t = 30 \text{ min} = 0.5 \text{ hr}$.

Time (min)	0	15	30	45	60	75	90	105	120	135		
P (in.)	0	0.2	0.6	1.1	1.5	1.8	2.0	2.2	2.3	2.5		
Time (hr)	1	2	3	4	5	6	7	8	9	10	11	12
Q (cfs)	100	200	500	800	700	550	450	250	150	100	75	0

Problem 1.29 utilizes the rainfall from Problem 1.28.

- If the gross rainfall of Problem 1.28 falls over a watershed with area of 3200 acres, find the volume that was left to infiltration (assume evaporation can be neglected) based on the volume under the hydrograph.
- Rework Example 1 from the case study on Jones Creek for Subbasin C (see PowerPoint on textbook website). The baseflow for Subbasin C is 200 cfs. In addition, the initial infiltration for the first hour is 1 in and a constant infiltration of 0.5 in thereafter. The storm hydrograph and rainfall hyetograph for Subbasin C are given in Figures P1–30a and P1–30b.

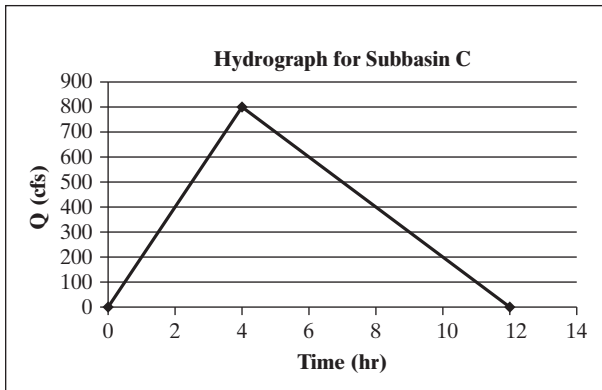


Figure P1-30a

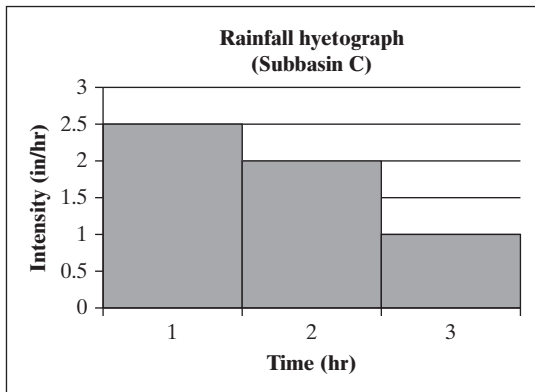


Figure P1-30b

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Chapter 2

Hydrologic Analysis



Typical flood scene of T.S. Allison in Houston, Texas.
(Photo courtesy of Dan Wallach.)

2.1 WATERSHED CONCEPTS

The **watershed** is the basic unit used in most hydrologic calculations relating to the water balance or computation of rainfall–runoff and other losses. The **watershed boundary** defines a contiguous area, such that the net rainfall or runoff over that area will contribute water to the outlet (Fig. 1–19(b)). Rain that falls outside the watershed boundary will generate runoff to some other outlet. A watershed boundary can be drawn from a topographic map by defining the ridgeline to the outlet, and runoff will generally travel from higher to lower elevation in a direction perpendicular to the elevation contours (see Fig. 1–19(b)).

Hydrologists are most often concerned with the amount of surface runoff generated within a watershed that becomes streamflow for a given input rainfall pattern. The main watershed characteristics that affect hydrologic response include the size, shape, slope, soil type, and storage within a watershed area. These issues are described in some detail in this chapter and form the basis for more detailed analyses in later chapters. The **hydrograph**, as presented earlier, is a plot of flow rate vs. time for a given location within a stream and represents the main hydrologic response function. Several

example watersheds were depicted in Figure 1–19(a), and a typical watershed with subareas depicted is shown in Figure 1–22.

Watersheds are often characterized by one main channel and by tributaries that drain into a main channel at one or more confluence points. The subarea of a tributary can be delineated by starting at the confluence and drawing a boundary along the subarea ridgeline. Larger watersheds can have many subareas that contribute runoff to a single outlet. An important watershed parameter is drainage area A , since it reflects the volume of water that can be generated from rainfall. Watershed length measures are depicted for subarea E in Figure 1–22. Channel length L is usually measured along the main channel from the outlet to the basin divide. Length to centroid L_c is measured along the main channel to a point nearest the centroid (center of mass) of the watershed. These two length parameters help determine watershed shape and are used extensively in unit hydrograph calculations discussed later (Section 2.3).

Another important physiographic parameter is **channel slope S** or **watershed slope S_0** , which reflects the rate of change of elevation with distance along the main channel or within an overland flow area. Both slope measures are used in performing unit hydrograph, flood routing, and time-of-travel calculations. Slope estimates are used in several of the unit hydrograph methods described later in this chapter. Field surveys or topographic maps, either paper or electronic, can be used to measure elevation changes so that watershed or channel slopes can be determined.

Soil types in a watershed are critical, as they determine infiltration rates that can occur for the area. Soil properties can vary significantly across a watershed area, and the USDA Natural Resources Conservation Service (NRCS) is responsible for developing soils maps to provide information on soil type, soil texture, and hydrologic soil groups. The three main soil classes are characterized by particle diameter d in mm, for sand, silt, and clay. Typical values are listed in Table 8–2 in Chapter 8. Soil texture is important in determining water-holding capacity and infiltration capacity of a soil layer. Thus, sands generally infiltrate water at a greater rate than do silts or clays. Of course, there can be mixtures of sizes, which can complicate the overall soil structure. The NRCS classified thousands of soils on the basis of runoff potential and grouped them into four hydrologic soil groups, A, B, C, and D. Type A infiltrates at the highest rate, D at the lowest. The relationship of soil type to infiltration capacity is presented in detail in Sections 2.8 and 2.9.

Land use and land cover, in the form of parking lots and urban development, can have profound effects on watershed response. In fact, many of the methods described later were developed to address urban development impacts in a watershed. For example, the **Rational Method** [see Eq. (1–13)] uses a coefficient C to reflect the runoff potential of a watershed. The value of C for commercial (0.75) is greater than residential (0.3), which is greater than forested (0.15), indicating that more intense development generates

greater rates of runoff for the same rainfall (see Sections 6.4 and 9.3). Urban development is also characterized by the percent imperviousness, or paved area, which can range from 50% to 90% for commercial compared to 20% to 40% for residential areas (see Table 2-1). Several of the unit hydrograph methods (Section 2.3) contain parameters that relate to urban land use effects.

Table 2-1 Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Land Use (Antecedent Moisture Condition II; $I_a = 0.25$)

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Cultivated land ¹				
Without conservation treatment	72	81	88	91
With conservation treatment	62	71	78	81
Pasture or range land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow				
Good condition	30	58	71	78
Wood or forest land				
Thin stand, poor cover, no mulch	45	66	77	83
Good cover ²	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.				
Good condition: grass cover on 75% or more of the area	39	61	74	80
Fair condition: grass cover on 50%–75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential ³				
Average lot size	Average % impervious ⁴			
1/8 ac or less	65	77	85	90
1/4 ac	38	61	75	83
1/3 ac	30	57	72	81
1/2 ac	25	54	70	80
1 ac	20	51	68	79
Paved parking lots, roofs, driveways, etc. ⁵	98	98	98	98
Streets and roads				
Paved with curbs and storm sewers ⁵	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89

¹For a more detailed description of agricultural land use curve numbers, refer to *National Engineering Handbook*, Section 4, "Hydrology," Chapter 9, Aug. 1972.

²Good cover is protected from grazing, and litter and brush cover soil.

³Curve numbers are computed assuming that the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur.

⁴The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

⁵In some warmer climates of the country, a curve number of 95 may be used.

Main channel and tributary characteristics can affect streamflow response in a variety of ways. As presented in Chapters 4 and 7, open-channel flow factors such as slope, cross-sectional area, Manning's roughness coefficient n , presence of obstructions, meander pattern, and channel condition can all contribute. Effects of channel geomorphology on flow patterns are covered in more detail in textbooks on geology and fluid mechanics. Floodplain analysis and floodplain mapping, presented in Chapters 7, 10, and 12, are based largely on an understanding of the nature of channel geomorphology as it relates to overland flow processes, which produces runoff that must be moved downstream by the channel. The shape, slope, and character of a floodplain will determine the volume and flow rate of water that can be safely handled during excessive events. The picture on the textbook cover shows the effects of a major out of bank flooding in Houston, TX, during Tropical Storm Allison in 2001. Flood problems occur in a watershed when either too much water is generated from a rainfall event or the channel is inadequate or not properly maintained to handle excessive overland flows.

Physiographic characteristics frequently used in hydrologic studies have been compiled for the USGS-EPA National Urban Studies Program (USGS, 1980) and are summarized below. Land use characteristics should be updated during the course of a hydrologic study to account for changes occurring in a watershed. Such physiographic information can be developed from maps describing land use, soils, topography, and storm drainage as well as from aerial photography. Fortunately, many advances in recent years in the area of geographic information systems (GIS), with the linkage of electronic maps and databases, have allowed this process to be greatly improved in time requirements and overall accuracy, as described in Chapter 10. A selected list of parameters follows below:

1. Total drainage area, in square miles
2. Impervious area in percentage of drainage area
3. Average basin slope determined and main conveyance slope at points 10% and 85% along the stream from the outlet to the divide
4. Hydraulic conductivity of the A horizon of the soil profile, in inches per hour
5. Hydrologic soil group and water capacity for soils (A, B, C, or D) according to NRCS methodology
6. Land use of the basins as a percentage of drainage area, including
 - (a) Rural and pasture
 - (b) Agricultural
 - (c) Low-density residential (0.5 to 2 acres per dwelling)
 - (d) Medium-density residential (3 to 8 dwellings per acre)
 - (e) High-density residential (9 or more dwellings per acre)
 - (f) Commercial

- (g) Industrial
- (h) Under construction (bare surface)
- (i) Idle or vacant land
- (j) Wetland
- (k) Parkland
- 7. Detention storage, in ac-ft of storage, and detention storage in ac-ft/acre of basin
- 8. Percent of watershed upstream from detention storage
- 9. Percent of area drained by a storm sewer system

2.2
UNIT
HYDROGRAPH
THEORY

Sherman (1932) originally advanced the theory of the unit hydrograph (UH), defined as “basin outflow resulting from 1.0 inch (1.0 mm) of direct runoff generated uniformly over the drainage area at a uniform rainfall rate during a specified period of rainfall duration.” An important point here is that UH is composed of 1.0 inch of direct runoff, which is equivalent to 1.0 inch of net rainfall for a given duration, D , and therefore all losses to infiltration must be subtracted before computations. Several assumptions inherent in the unit hydrograph approach tend to limit its application for any given watershed (Johnstone and Cross, 1949):

1. Rainfall excesses of equal duration are assumed to produce hydrographs with equivalent time bases regardless of the intensity of the rain.
2. Direct runoff ordinates for a storm of given duration are assumed directly proportional to rainfall excess volumes. Thus, twice the rainfall produces a doubling of hydrograph ordinates.
3. The time distribution of direct runoff is assumed independent of antecedent precipitation.
4. Rainfall distribution is assumed to be the same for all storms of equal duration, both spatially and temporally.

The classic statement of unit hydrograph theory can be summarized briefly: The hydrologic system is linear and time invariant (Dooge, 1973). The linear assumption implies that complex storm hydrographs can be produced by adding up individual unit hydrographs, adjusted for rainfall volumes and added and lagged in time. For example, a 2-inch rainfall in X hours will produce double the response of 1 inch falling in X hours over a given watershed. While the assumptions of linearity and time invariance are not strictly correct for every watershed, we adopt them as long as they are useful. Nonlinear examples exist in open-channel flow, laboratory runoff models, and actual watersheds.

Derivation of Unit Hydrographs: Gaged Watersheds

A typical storm hydrograph and rainfall hyetograph for a drainage basin are shown in Figure 2-1(a). The total storm hydrograph is a simple plot of flow or discharge (cfs or cms) vs. time. The hydrograph is characterized by

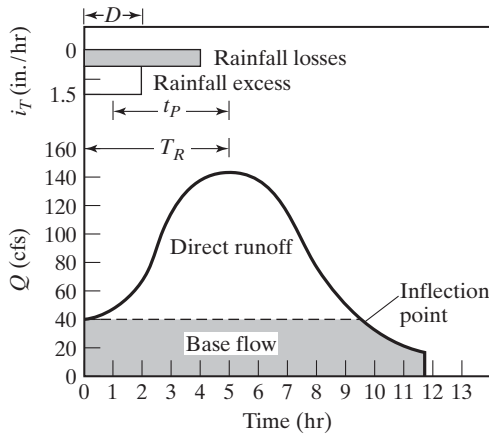
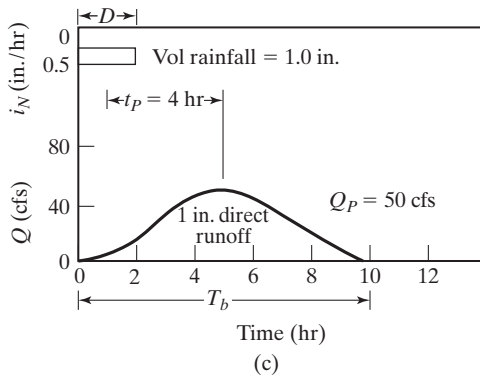
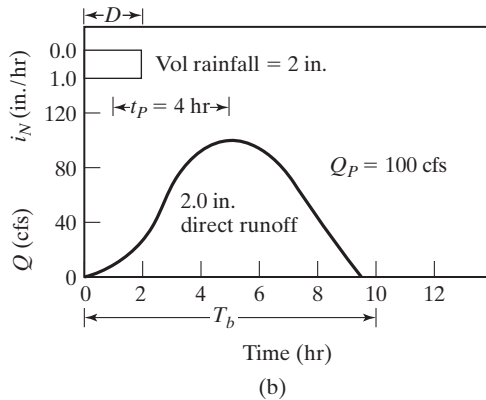


Figure 2-1

Unit hydrograph determination. (a) Total storm hydrograph. (b) Hydrograph minus baseflow, rainfall minus losses. (c) Hydrograph adjusted to be a 2-hr unit hydrograph.



a rising limb, a crest segment, and a recession curve (see Fig. 1–26). Timing parameters such as **duration** D of rainfall excess and **time to peak**, t_p , are also illustrated. The main timing aspects of the hydrograph can be characterized by the following parameters:

1. **Duration** of rainfall excess (D): the time from start to finish of rainfall excess.
2. **Lag time** (t_p): the time from the center of mass of rainfall excess to the peak of the hydrograph.
3. **Time of rise** (T_R): the time from the start of rainfall excess to the peak of the hydrograph.
4. **Time of concentration** (t_c): the time for a wave (of water) to propagate from the most distant point in the watershed to the outlet. One estimate is the time from the end of net rainfall to the inflection point of the hydrograph.
5. **Time base** (T_b): the total duration of the DRO hydrograph.

If rainfall–runoff data exist, it is useful to develop unit hydrographs from measured rainfall events and streamflow from gages in a defined watershed. The following general rules should be observed in developing unit hydrographs from gauged watersheds:

1. Storms should be selected with a simple structure with relatively uniform spatial and temporal distributions.
2. Watershed sizes should generally fall between 1.0 and 100 mi² in modern watershed analysis.
3. Direct runoff should range from 0.5 to 2.0 in.
4. Duration of rainfall excess D should be approximately 25% to 30% of lag time t_p .
5. A number of storms of similar duration should be analyzed to obtain an average unit hydrograph for that duration.
6. Step 5 should be repeated for several rainfalls of different duration.

The following are the essential steps for developing a unit hydrograph from a single storm hydrograph (see Fig. 2–1 and Example 2–1):

1. Analyze the hydrograph and separate base flow.
2. Measure the total volume of DRO under the hydrograph and convert this to inches (mm) over the watershed.
3. Convert total rainfall to rainfall excess through infiltration methods, such that rainfall excess = DRO, and evaluate duration D of the rainfall excess that produced the DRO hydrograph.
4. Divide the ordinates of the DRO hydrograph (Fig. 2–1(b)) by the volume in inches (mm) and plot these results as the unit hydrograph

for the basin (Fig. 2–1(c)). The time base T_b is assumed constant for storms of equal duration and thus it will not change.

5. Check the volume of the unit hydrograph to make sure that it is 1.0 in. (1.0 mm), and graphically adjust ordinates as required.

DETERMINATION OF UNIT HYDROGRAPH

EXAMPLE 2–1

Convert the direct runoff hydrograph shown in Figure E2–1a into a 2-hr UH. (The UH is labeled with its duration.) Note that the duration of net rainfall is 2 hr. The total rainfall hyetograph is given in the figure, and the infiltration loss for the storm was 0.5 in./hr as a constant value. The base flow in the channel was 100 cfs (constant). What are t_p and T_b for the storm?

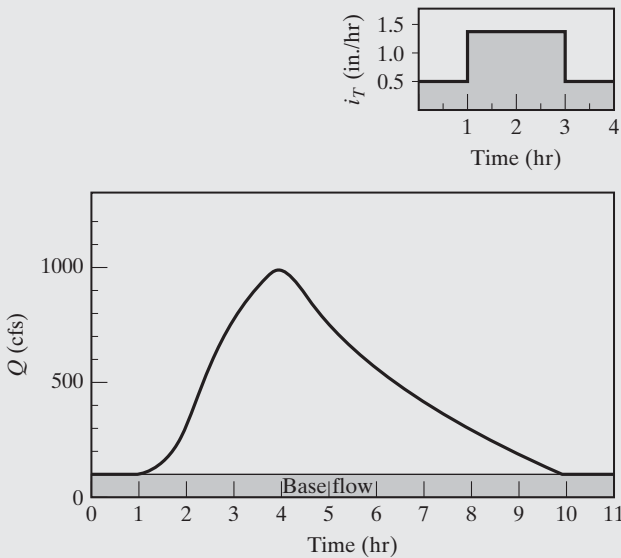


Figure E2–1a
Runoff hydrograph
with baseflow and
rainfall.

First we find the net rainfall. After subtracting the infiltration, we plot the rainfall excess hyetograph shown in Figure E2–1b. This represents 2.0 in. of rainfall, or 1 in./hr for 2 hr. Then we subtract base flow from all the flow values. Finally, the hydrograph must be converted to 1 in. of direct runoff over the watershed, or 0.5 in./hr of rain for 2 hr. To do so,

SOLUTION

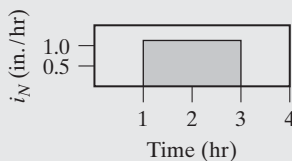


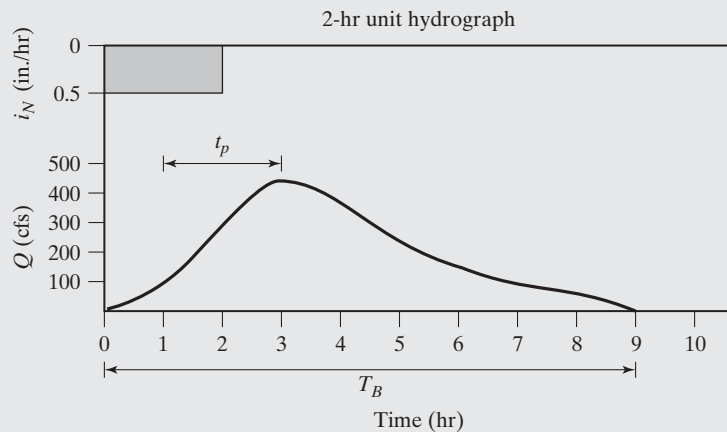
Figure E2–1b
Resulting
hyetograph.

we take each ordinate minus its base flow and divide it by 2. This entire procedure is tabulated as follows.

Time (hr)	Q (cfs)	Q-BF (cfs)	2-hr UH, Q
0	100	0	0
1	100	0	0
2	300	200	100
3	700	600	300
4	1000	900	450
5	800	700	350
6	600	500	250
7	400	300	150
8	300	200	100
9	200	100	50
10	100	0	0
11	100	0	0

The 2-hr UH graphs as shown in Figure E2-1c. T_b , the time base of the storm, is 9 hr, and the time to peak t_p measured from the center of mass of rainfall is 2 hr.

Figure E2-1c
2-hr UH.



Because of assumptions of linearity inherent in the UH development, care must be used in applying UHs under conditions that tend to violate linearity. If intensity variations are large over the long-duration storm, assumptions of linearity may be violated. Also, if storage effects in the watershed are important, assumptions may be violated. For large watersheds (above 100–500 mi²), it is usually advisable to divide the watershed into a number of subareas so that individual UHs can be developed for each. Typically, UH should not exceed areas of 5 to 10 sq mi.

Amount of runoff and areal distribution of runoff can cause variations in the shape of the hydrograph. For large basins, it is usually best to develop UHs for subareas and then add and lag, or convolute, them together to generate an overall hydrograph. Peaks of UHs derived from very small events are often lower than those derived from larger storms because of differences in interflow and channel flow times. Assumptions of linearity can usually be verified by comparing hydrographs from storms of various magnitudes. If nonlinearity does exist, then derived UHs should be used only for generating events of similar magnitude, and caution should be exercised in using unit hydrographs to extrapolate to extreme events. Despite these limitations, when unit hydrographs have been used in conjunction with modern flood routing methods, resulting flood prediction for small and large basins is quite accurate. Detailed applications of the unit hydrograph are presented later in this chapter and in Chapters 5 and 9.

S-Curve Method

A UH for a particular watershed is defined for a specific duration D of rainfall excess (RF_{net}). The linear property of the UH can be used to generate a UH of larger or smaller duration. For example, given a 1-hr UH for a particular watershed, a UH resulting from a 2-hr unit storm can be developed by adding two 1-hr UHs, the second one lagged by 1 hr, adding together the ordinates, and dividing the result by 2. In this way, the 1 in. of rainfall in 1 hr has been distributed uniformly over 2 hr in deriving the 2-hr UH (Fig. 2-2). This lagging procedure is restricted to integer multiples of the original duration. In the same way, the 3-hr UH results from adding and lagging three together and dividing the ordinates by 3 (see Example 2-3).

The **S-curve method** allows construction of a UH of any duration. Assume that a UH of duration D is known and that we wish to generate a UH for the same watershed with duration D' . The first step is to generate the S-curve hydrograph by adding a series of UHs of duration D , each lagged by time period D . This corresponds to the runoff hydrograph resulting from a continuous rainfall excess intensity of $1/D$ in./hr, where D is measured in hours. Note that an equilibrium hydrograph (S-curve) results

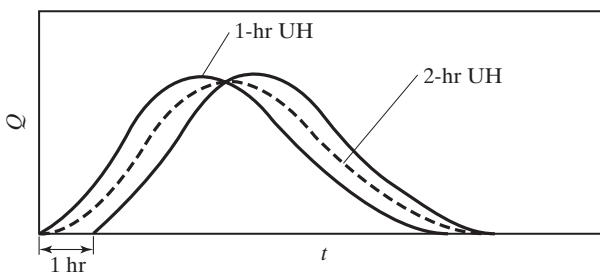
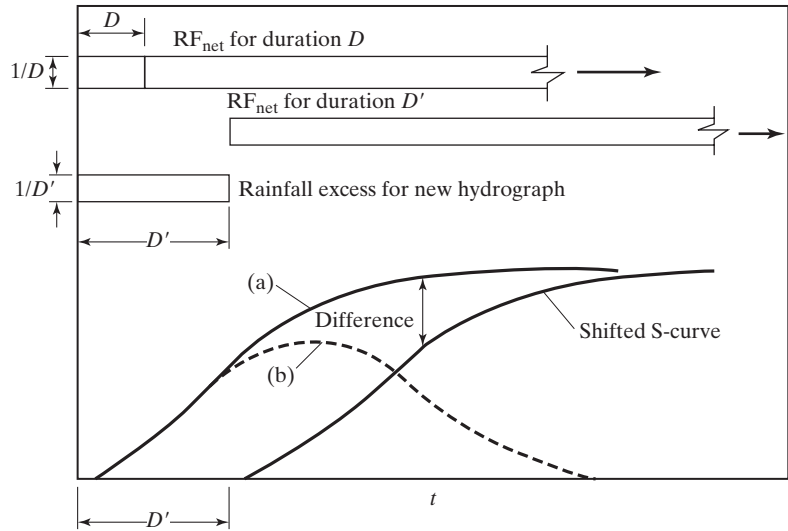


Figure 2-2

Graphical representation of unit hydrograph lagging (from a 1-hr UH to a 2-hr UH).

**Figure 2-3**

Graphical representation of the S-curve hydrograph method.

from this addition of many UHs, each lagged by D hr (see Fig. 2-3), and is equivalent to a continuous rainfall intensity falling until inflow and outflow are equal.

By shifting the S-curve in time by D' hr and subtracting ordinates between the two S-curves, the resulting hydrograph (Fig. 2-3, curve b) must be due to rainfall of $1/D$ in./hr that occurs for D' hr. Thus, to convert curve b to a UH, we must multiply all the hydrograph ordinates by D/D' resulting in the UH of duration D' . A homework problem explores these calculations in detail. The procedure is ideally suited for an Excel spreadsheet.

Unit Hydrograph Convolution

The real importance of the UH approach is demonstrated in the development of storm hydrographs due to an actual rainfall event over a watershed. UH ordinates U_j are multiplied by rainfall excess P_n and added and lagged in a sequence to produce a resulting storm hydrograph, which ideally can be done using available Excel spreadsheets. Hydrograph convolution, the procedure of deriving a storm hydrograph from a multiperiod rainfall excess, is illustrated in Example 2-2. For actual watersheds, these calculations are almost always done in a hydrologic model such as HEC-HMS (see Chapter 5).

Each unit hydrograph is added at the time corresponding to the rainfall spike that produced a response. Base flow can be added to produce a

realistic storm hydrograph if base flow values are available for the watershed under study. Care should be taken in this calculation to ensure that the time increments of rainfall excess correspond exactly to the duration of the UH. For example, 1-hr time increments should be used with the 1-hr UH. The governing equation for the storm hydrograph in discrete form is called the **convolution equation**:

$$Q_n = \sum_{i=1}^n P_i U_{n-i+1}, \quad (2-1)$$

or

$$Q_n = P_n U_1 + P_{n-1} U_2 + P_{n-2} U_3 + \cdots + P_1 U_n,$$

where Q_n is the storm hydrograph ordinate, P_i is rainfall excess, and U_j ($j = n - i + 1$) is the unit hydrograph ordinate. Periods of no rainfall can also be included, as shown in Example 2-2.

STORM HYDROGRAPH FROM THE UNIT HYDROGRAPH

EXAMPLE 2-2

Given the rainfall excess hyetograph and the 1-hr UH below, derive the storm hydrograph for the watershed using hydrograph convolution (add and lag). Compute the resulting storm hydrograph and assume no losses to infiltration or evapotranspiration.

Equation (2-1) states

SOLUTION

$$Q_n = P_n U_1 + P_{n-1} U_2 + P_{n-2} U_3 + \cdots + P_1 U_n.$$

Here net rainfall is provided,

$$P_n = [0.5, 1.0, 1.5, 0.0, 0.5] \text{ in.},$$

from the rainfall hyetograph. Using intervals of 1 hr, we have unit hydrograph ordinates

$$U_n = [0, 100, 320, 450, 370, 250, 160, 90, 40, 0] \text{ cfs}$$

from the unit hydrograph. By Eq. (2-1),

$$Q_0 = (0.5)(0) = 0,$$

$$Q_1 = (1.0)(0) + (0.5)(100) = 50 \text{ cfs},$$

$$Q_2 = (1.5)(0) + (1.0)(100) + (0.5)(320) = 260 \text{ cfs}, \dots$$

The value derived from these calculations can be tabulated as in Table E2-2.

Q_n is equal to the sum across the row for time n . The resulting storm hydrograph, Q_n , is shown in Table E2-2. Spreadsheets are available on the textbook website and Appendix E and should be used to solve many of the homework problems in Chapter 2.

Table E2-2

Time (hr)	P_1U_n	P_2U_n	P_3U_n	P_4U_n	P_5U_n	Q_n
0	0					0
1	50	0				50
2	160	100	0			260
3	225	320	150	0		695
4	185	450	480	0	0	1115
5	125	370	675	0	50	1220
6	80	250	555	0	160	1045
7	45	160	375	0	225	805
8	20	90	240	0	185	535
9	0	40	135	0	125	300
10		0	60	0	80	140
11			0	0	45	45
12				0	20	20
13					0	0

The reverse procedure using matrix methods allows us to determine a UH from a direct storm hydrograph produced by a multiperiod rainfall excess. For a four-period rainfall (Fig. 2-4), the following equations result:

$$\begin{aligned}
 Q_1 &= P_1U_1, \\
 Q_2 &= P_2U_1 + P_1U_2, \\
 Q_3 &= P_3U_1 + P_2U_2 + P_1U_3, \\
 Q_4 &= P_4U_1 + P_3U_2 + P_2U_3 + P_1U_4, \\
 Q_5 &= P_4U_2 + P_3U_3 + P_2U_4 + P_1U_5, \\
 Q_6 &= P_4U_3 + P_3U_4 + P_2U_5 + P_1U_6, \\
 Q_7 &= P_4U_4 + P_3U_5 + P_2U_6 + P_1U_7, \\
 Q_8 &= P_4U_5 + P_3U_6 + P_2U_7, \\
 Q_9 &= P_4U_6 + P_3U_7, \\
 Q_{10} &= P_4U_7.
 \end{aligned} \tag{2-2}$$

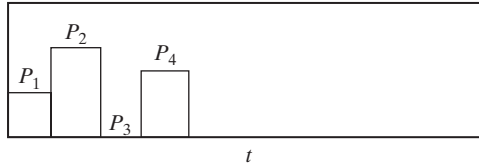
The relationship between the number of storm hydrograph periods n , the UH ordinates j , and the rainfall excess periods i is

$$n = j + i - 1.$$

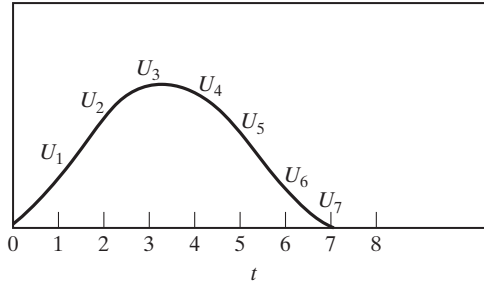
The general equation [Eq. (2-2)] can be expressed in a more compressed form as

$$[Q] = [P][U], \tag{2-3}$$

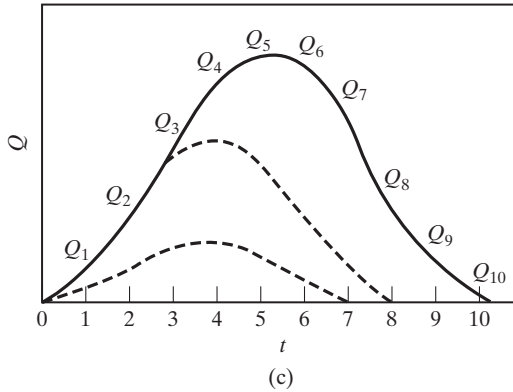
where $[P]$ is a matrix that can be solved for $[U]$. Such a solution requires the inverse matrix $[P]^{-1}$, which must be a square matrix with a nonzero



(a)



(b)



(c)

Figure 2-4

Graphical representation of a unit hydrograph for a multiperiod storm

determinant. The matrix $[P]$ does not meet this condition, but by using the transpose matrix $[P^T]$, a square symmetrical matrix $[P^T P]$ can be generated. The matrix $[P^T P]$ can then be used to obtain

$$[P^T P][U] = [P^T][Q]. \quad (2-4)$$

This equation generates simultaneous normal equations for a least-squares solution. Equation (2-4) can be solved explicitly for $[U]$ using

$$[U] = [P^T P]^{-1} [P^T][Q]. \quad (2-5)$$

Equation (2-3) can be solved using the Gauss elimination method of solving simultaneous equations or Excel's matrix solver tools. As with any numerical procedure, under certain conditions the error in the values of $[U]$

can grow rapidly, and unrealistic or oscillating values can result, especially for later ordinates of the UH. Note that the decomposition procedure is rarely used, and it is usually better to develop UHs from simple storms, if possible.

EXAMPLE 2-3**UNIT HYDROGRAPH EXAMPLE**

Convert the 1-hr unit hydrograph (UH) given below to a 3-hr UH and find the area of the watershed.

Time (hr)	UH (1 hr)
0	0
1	25
2	50
3	100
4	80
5	60
6	40
7	20
8	0

SOLUTION

First, we graph the 1-hr unit hydrograph.

To transform a 1-hr UH to a 3-hr UH, we must first lag the 1-hr UH twice, thus producing three 1-hr unit hydrographs over 3 hr.

Time (hr)	UH (1-hr)	Lagged 1-hr UH	Lagged 1-hr UH
0	0	0	0
1	25	0	0
2	50	25	0
3	100	50	25
4	80	100	50
5	60	80	100
6	40	60	80
7	20	40	60
8	0	20	40
9	0	0	20
10	0	0	0

After we have lagged the UH, we must add the three unit hydrographs together and divide by 3 in our case to adjust to 1.0 in. of direct runoff.

Sum of all 1-hr UH	3-hr UH [(added 3 1-hr UH)/3]
0	0
25	8.3
75	25.0
175	58.3
230	76.7
240	80.0
180	60.0
120	40.0
60	20.0
20	6.7
0	0

The new peak flow is 80 cfs at $t = 5$ hr. To find the area of the watershed, which is usually known, add up the ordinates for each hour of the hydrograph and multiply by the incremental hour.

In our case, the total flow is 375 cfs (1 hr).

To find the area: (note 1 acre-inch = 1.008 cfs-hr)

$$375 \text{ cfs (1 hr)} = 372 \text{ acre-inch}$$

A number of equations have been developed over the years to predict many of the above unit hydrograph parameters as a function of measurable watershed characteristics, and several of the more popular approaches are presented in this section. With the increasing use of computer models in hydrology and GIS methods, the synthetic methods have generally replaced the gauged-based method for most watersheds, especially since many watersheds are not gaged.

Methods for deriving UH for ungaged watersheds have evolved based on theoretical or empirical formulas relating hydrograph peak flow and timing to watershed characteristics. These are usually referred to as **synthetic unit hydrographs** (UH) and offer the hydrologist or engineer a multitude of methods for developing a UH for a particular basin. Most UH methods were developed in the period from 1932 to 1970, and they still provide one of the most useful and accurate approaches for hydrologic prediction for a given rainfall event.

Synthetic UHs, once developed for a watershed area, can be used with historical or design rainfalls (see Example 2–2) to produce storm hydrographs at the outlet of the watershed. As the watershed changes over time, the UH can be updated to better represent land use and channel alterations. Synthetic UHs developed along two main lines of thought; one assumed that each watershed had a unique UH related to specific watershed characteristics, and the second assumed that all UHs could be represented by a single

2.3 SYNTHETIC UNIT HYDROGRAPH DEVELOPMENT

family of curves or a single equation. However, the formulas all have certain limiting assumptions and should be applied to new areas with extreme caution. Some calibration to adjacent watersheds where streamflow gages exist should be attempted, if possible.

The first line of development was based on the rational method modified to include the travel time to the outlet for a particular watershed. Clark (1945) assumed that watershed response would be given by routing the time–area relationship through an element of linear storage (see Eq. 2–23), which tends to attenuate and time-lag the hydrograph. The Clark unit hydrograph is contained in HEC-HMS and is presented in detail later in this section and in Table 5–10. The second approach to UH development assumed mathematical representations for the shape of the UH. A useful approach was advanced by the Soil Conservation Service (SCS, 1964, 1986), based on measurements from thousands of small watershed areas, which represented a dimensionless UH of discharge vs. time by a gamma function (discussed later). Since volume is fixed, only one parameter is required to determine the entire UH, either the t_p or the peak flow rate, Q_p .

In the late 1950s, both approaches began to converge after the important contributions of O’Kelly (1955) and Nash (1958, 1959). O’Kelly’s work was based on replacing the time–area curve by a UH in the shape of an isosceles triangle. Empirical expressions were necessary to transform the methods of Clark (1945) into usable UH techniques for actual basins. Johnstone and Cross (1949) proposed one of the first relationships for t_c , the time of concentration in hours, defined earlier in the chapter.

$$t_c = 5.0(L/\sqrt{S})^{0.5}, \quad (2-6)$$

and for **storage delay time**, K , in hours,

$$K = 1.5 + 90(A/LR), \quad (2-7)$$

where L is the length of the main stream in mi, A is the area in mi^2 , S is the slope in ft/mi , and R is an overland slope factor.

Hydrologists have realized that a number of parameters are important in determining the shape and timing of the UH for a watershed. The lag or time to peak t_p and the time of concentration t_c are often used (Fig. 2–1(a)). The time of rise T_R measured from time zero to the hydrograph peak is sometimes used. The time base T_b of the hydrograph is included to define the duration of direct runoff. These timing parameters must be statistically or theoretically related to watershed characteristics in developing a synthetic UH.

The discharge parameter most often used is the peak discharge Q_p . A **routing parameter** K is sometimes included when the hydrograph has been routed through a linear reservoir with storage delay time K . This tends to delay and attenuate the input hydrograph. Watershed parameters of most concern include area A , main channel length L , length to watershed centroid L_c , and slope of main channel S (see Section 2.1 and Fig. 1–22).

Assumptions and Modifying Factors

Most synthetic UH methods assume that the UH of a basin represents the integrated effect of size, slope, shape, and storage characteristics. As long as these factors are constant between two basins and do not vary with time, the unit response will be identical for the two basins. For two basins of the same size, if the slope of one is greater or if the basin shape of one is more concentrated (if the length/width ratio is lower), the shape of the hydrograph will shift, as shown by the shift from *a* to *b* in Figure 2–5. Storage characteristics of the basin relate to slope, soil type, topography, channel resistance, and shape. If upstream reservoir storage is combined with downstream channelization, the UH may shift to curve *b* due to a more rapid rise time from concrete channels and less infiltration loss from urban development. Finally, if an elongated basin were fully developed, one might expect a shift to curve *c*, where time of rise is much shorter and peak flow is much greater than in the original UH, *a*. Such a case is shown in Figure 2–6 for a fully developed watershed, Brays Bayou in Houston, Texas. Original synthetic hydrograph methods generally do not consider urbanization effects, but more modern empirical formulas usually account for urban channels, percent impervious (paved) area, or percent storm-sewered area.

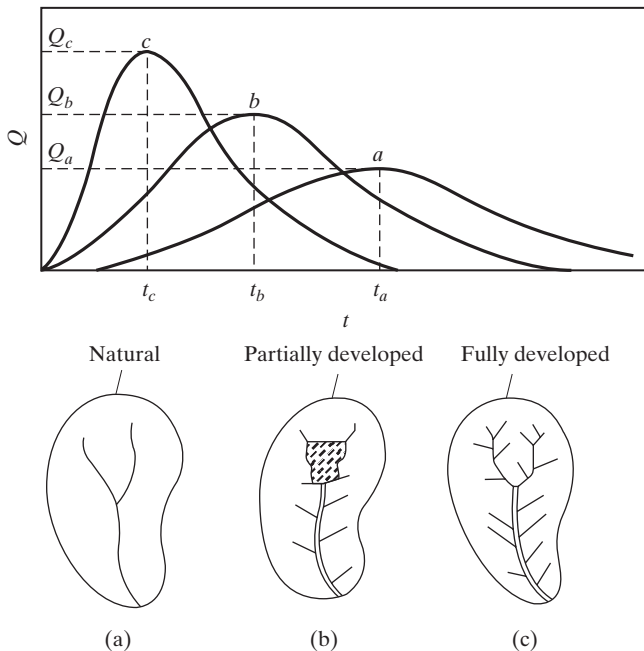
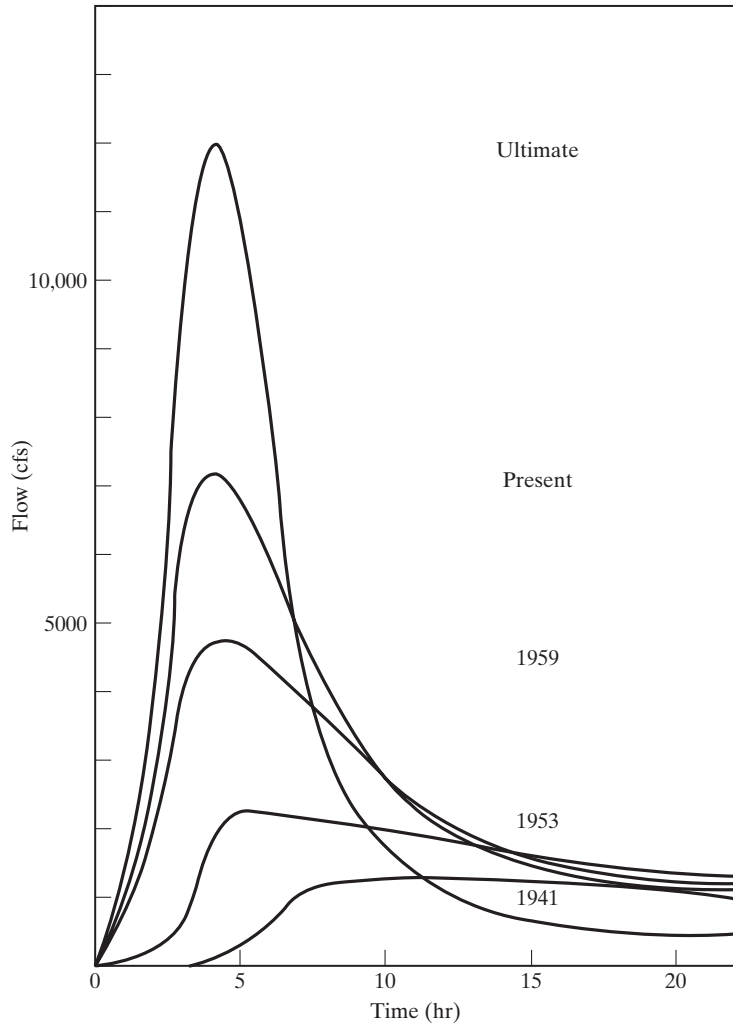


Figure 2–5

Modifying factors on unit hydrographs. (a) Natural watershed development, represented by curve *a* in the bottom part of the figure. (b) Partial development, represented by curve *b*. (c) Fully developed watershed, represented by curve *c*.

Figure 2-6

Changes in land use cause changes in the shape of the Brays Bayou UH.



Most of the methods for synthetic UHs relate lag time t_p or time of rise T_R of the hydrograph to measures of the length of the main channel and shape of the basin. Some methods also relate timing to the inverse of the slope of the main channel or land. Thus, the longer the basin and the smaller the slope, the greater the time of rise of the hydrograph, as expected. Figure 1-19(a) (Chapter 1) shows the general effect of watershed shape and land use (i.e., elongated vs. concentrated) on UH response.

A second relation is usually presented between peak flow Q_p and area of basin, and between Q_p and the inverse of the t_p or T_R of the hydrograph, thus indicating that larger areas produce higher Q_p . From continuity, the higher the peak flow, the smaller t_p must be to keep the volume of the unit

hydrograph constant with 1.0 in. of direct runoff. This is easily seen by referring to Figure 2-5, where curves *a*, *b*, and *c* are all unit hydrographs. The following sections describe a few of the more popular synthetic unit hydrograph methods. Specific areas of the United States have developed their own empirical methods that are similar to the standard methods, but based on local watershed data.

Snyder's Method

Snyder (1938) was the first to develop a synthetic UH based on a study of watersheds in the Appalachian Highlands. In basins ranging from 10 to 10,000 mi², Snyder's relations are

$$t_p = C_t(LL_c)^{0.3}, \quad (2-8)$$

where

- t_p = basin lag (hr),
- L = length of the main stream from the outlet to the divide (mi),
- L_c = length along the main stream to a point nearest the watershed centroid (mi),
- C_t = coefficient usually ranging from 1.8 to 2.2 (C_t has been found to vary from 0.4 in mountainous areas to 8.0 along the Gulf of Mexico),

$$Q_p = 640 C_p A / t_p, \quad (2-9)$$

where

- Q_p = peak discharge of the unit hydrograph (cfs),
- A = drainage area (mi²),
- C_p = storage coefficient ranging from 0.4 to 0.8, where larger values of C_p are associated with smaller values of C_t ,

$$T_b = 3 + t_p/8, \quad (2-10)$$

where T_b is the time base of the hydrograph, in days. For small watersheds, Equation (2-10) should be replaced by multiplying t_p by a value that varies from 3 to 5 as a better estimate of T_b . Equations (2-8), (2-9), and (2-10) define points for a unit hydrograph produced by an excess rainfall of duration $D = t_p/5.5$. For other rainfall excess durations D' , an adjusted formula for t_p becomes

$$t'_p = t_p + 0.25(D' - D), \quad (2-11)$$

where t'_p is the adjusted lag time (hr) for duration D' (hr). Once the three quantities t_p , Q_p , and T_b are known, the UH can be sketched so that the area under the curve represents 1.0 in. of direct runoff from the watershed.

Snyder's method is a popular method because of its simplicity. Caution should be used in applying Snyder's method to a new area without first deriving

coefficients for gaged streams in the general vicinity of the problem basin. The coefficients C_t and C_p have been found to vary considerably from one region to another. Example 2-4 illustrates Snyder's method. Width equations at 50% and 75% of Q_p are presented in Example 2-4 to help sketch the Snyder UH.

EXAMPLE 2-4**SNYDER'S METHOD**

Use Snyder's method to develop a UH for the area of 100 mi² described below. Sketch the approximate shape. What duration rainfall does this correspond to?

$$C_t = 1.8, \quad L = 18 \text{ mi},$$

$$C_p = 0.6, \quad L_c = 10 \text{ mi}$$

SOLUTION

The UH is sketched in Figure E2-4. Note that width equations for W_{50} and W_{75} are given there to help shape the UH.

By Equation (2-8),

$$t_p = C_t(LL_c)^{0.3},$$

$$t_p = 1.8(18 \cdot 10)^{0.3} \text{ hr},$$

$$t_p = 8.6 \text{ hr}.$$

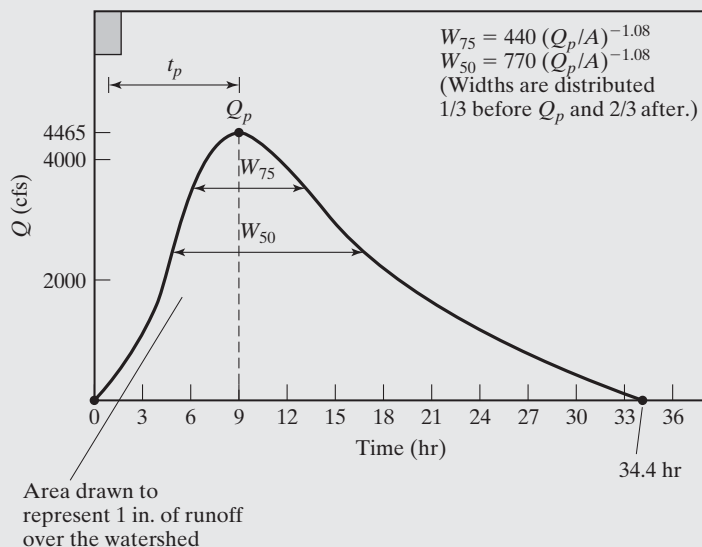
By Equation (2-9),

$$Q_p = 640(C_p)(A)/t_p,$$

$$= 640(0.6)(100)/8.6,$$

$$Q_p = 4465 \text{ cfs}.$$

Figure E2-4
Typical Snyder unit hydrograph with special relationships indicated.



Since this is a small watershed,

$$\begin{aligned}T_b &\approx 4t_p = 4(8.6) \text{ hr}, \\T_b &= 34.4 \text{ hr}.\end{aligned}$$

And the duration of rainfall

$$\begin{aligned}D &= t_p/5.5 \text{ hr}, \\&= 8.6/5.5 \text{ hr}, \\D &= 1.6 \text{ hr}.\end{aligned}$$

Finally, the hydrograph should be smoothed to represent 1.0 in. of direct runoff.

SCS Methods

The methods developed by the Soil Conservation Service (SCS, 1957, 1964) are based on a **dimensionless hydrograph**, developed from a large number of UHs from gaged watersheds ranging in size and geographic location (Fig. 2-7). The SCS is now called the Natural Resources Conservation Service (NRCS). More details and examples on SCS methods can be found in McCuen (2005). The earliest method assumed a hydrograph represented as a simple triangle (Fig. 2-7), with rainfall duration D (hr), time of rise T_R (hr), time of fall B (hr), and peak flow Q_p (cfs). The volume of direct runoff is

$$\text{Vol} = \frac{Q_p T_R}{2} + \frac{Q_p B}{2} \quad (2-12)$$

or

$$Q_p = \frac{2 \text{ Vol}}{T_R + B}. \quad (2-13)$$

From a review of a large number of hydrographs, it was found that

$$B = 1.67 T_R. \quad (2-14)$$

Therefore, Equation (2-12) becomes, for 1.0 in. of rainfall excess,

$$\begin{aligned}Q_p &= \frac{0.75 \text{ Vol}}{T_R} \\&= \frac{(0.75)(640) A(1.008)}{T_R} \\&= \frac{484 A}{T_R},\end{aligned} \quad (2-15)$$

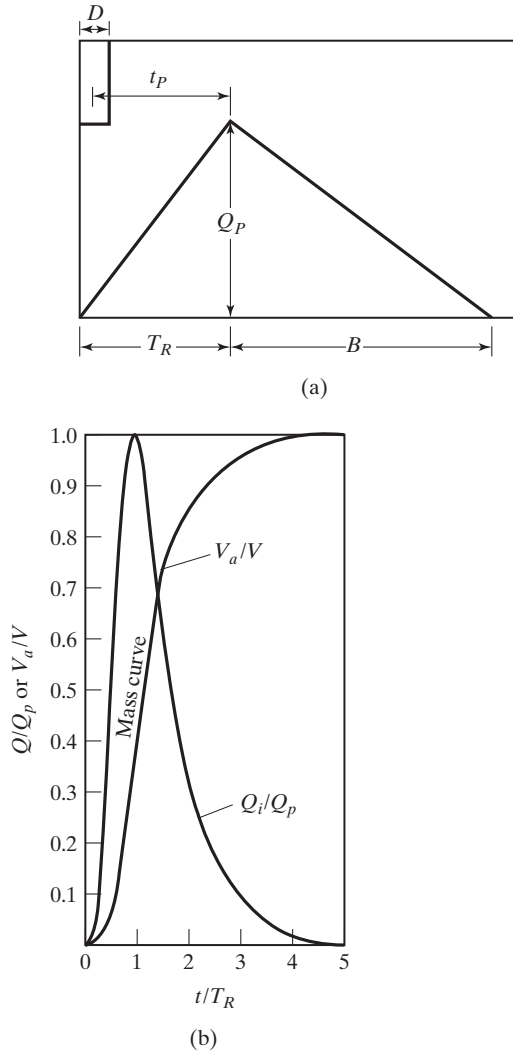
$$= \frac{484 A}{T_R}, \quad (2-16)$$

where

A = area of basin (sq mi),
 T_R = time of rise (hr).

Figure 2-7

(a) SCS triangular unit hydrograph.
 (b) SCS dimensionless unit hydrograph (SCS 1964).



Capece et al. (1984) found that a factor as low as 10–50 holds for flat, high-water-table watersheds rather than the value 484 presented here. Mc Cuen indicates a possible range from 300 to 600.

From Figure 2-7 it can be shown that

$$T_R = \frac{D}{2} + t_p, \quad (2-17)$$

where

D = rainfall duration (hr),

t_p = lag time from centroid of rainfall to Q_P (hr).

Lag time t_p is estimated from any one of several empirical equations used by the SCS, and the one that is often reported is (note that units must be carefully followed in the SCS method)

$$t_p = \frac{L^{0.8}(S + 1)^{0.7}}{1900\sqrt{y}} \quad (2-18)$$

where

t_p = lag time (hr),
 L = length to divide (ft),
 y = average watershed slope (in percent),

$$S = 1000/\text{CN} - 10 \text{ (in.)}, \quad (2-19)$$

CN = curve number for various soil/land use (see Table 2-1).

The SCS dimensionless UH can be used to develop a curved hydrograph, using the same t_p and Q_p as the triangular hydrograph in Figure 2-7 (see Example 2-6). The SCS also found that time of concentration t_c was equal to 1.67 times the lag time above.

SCS (1964) runoff estimates assume a relationship between accumulated total storm rainfall P , runoff Q , and infiltration plus initial abstraction ($F + I_a$). Based on SCS watershed studies

$$I_a = 0.2S, \quad (2-20)$$

It is also assumed that

$$F/S = Q/P_e, \quad (2-21)$$

where F is infiltration occurring after runoff begins, S is **potential abstraction**, Q is direct runoff (inches), and P_e is effective storm runoff ($P - I_a$). Given $F = (P_e - Q)$ and $P_e = (P - I_a) = (P - 0.2S)$ based on data from small watersheds, one can show

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}. \quad (2-22)$$

The SCS method uses the runoff curve number CN, related to potential abstraction S by $\text{CN} = 1000/(S + 10)$, or $S \text{ (in.)} = (1000)/\text{CN} - 10$. Figure 2-8 presents SCS Equation (2-22) in a graphical form for a range of CN values and rainfalls.

Runoff curve numbers for selected land uses are presented in Table 2-1, where hydrologic soil group A is sandy and well drained, group B is sandy loam, group C is clay loam or shallow sandy loam, and group D has a poorly drained, heavy plastic clay that swells when wet. Group A has the highest infiltration capacity and Group D the lowest. The CN values in Figure 2-8 assume normal antecedent moisture condition II, and other antecedent moisture conditions and effects of urbanization can be developed using the SCS report, *Urban Hydrology for Small Watersheds* (SCS, 1986). For a

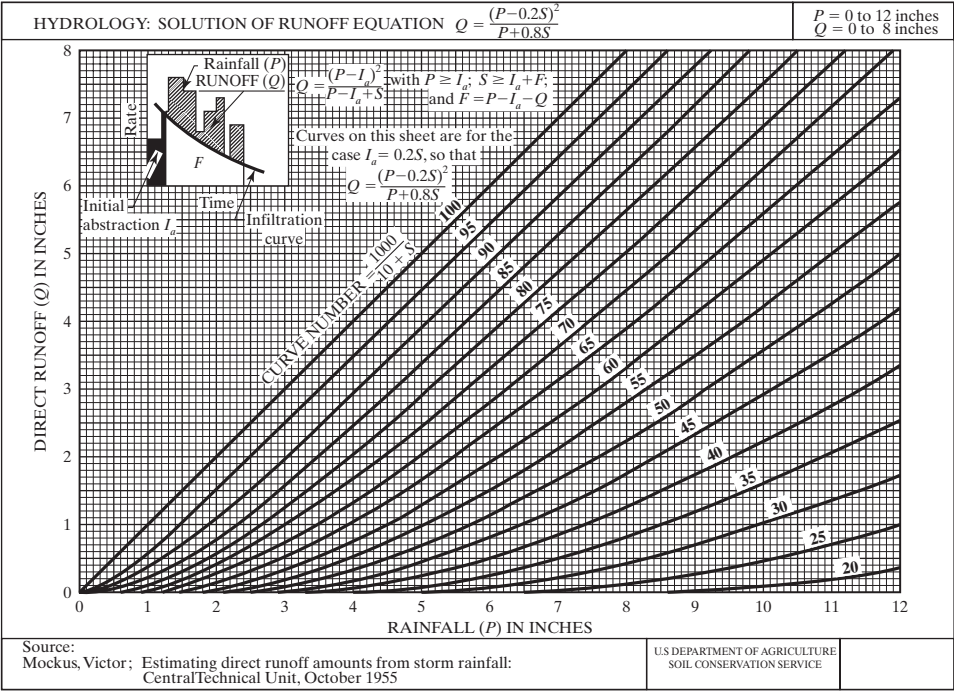


Figure 2-8
Graphical solution of rainfall-runoff equation.

watershed made up of several soil types and land uses, a composite CN can be calculated (Example 2-5). The TR-20 and TR-55 computer programs are the adaptations of the SCS methods for nonurban and urban areas, respectively. McCuen (2005) provides more detailed coverage with examples on SCS methods.

EXAMPLE 2-5

SCS CURVE-NUMBER METHOD

A watershed is 40% wooded (good condition) and 60% residential (1/4-ac lots). The watershed has 50% soil group B and 50% soil group C. Determine the runoff volume if the rainfall is 7 in. Assume antecedent moisture condition number II (Table 2-1).

SOLUTION

Land Use	Soil Group	Fraction of Area	CN
Wooded	B	$0.4(0.5) = 0.2$	55
	C	$0.4(0.5) = 0.2$	70
Residential	B	$0.6(0.5) = 0.3$	75
	C	$0.6(0.5) = 0.3$	83

The weighted CN is

$$\text{CN} = 0.2(55) + 0.2(70) + 0.3(75) + 0.3(83) \text{ or}$$

$$\text{CN} = 11 + 14 + 22.5 + 24.9 = 72.4$$

or, using $\text{CN} = 72$, runoff volume is 3.9 in for the given rainfall (Fig. 2–8).

While SCS methods are used extensively in engineering practice, and a PC version for urban watersheds (TR-55) is available (SCS, 1986), the methods have some weaknesses, as pointed out by Capece et al. (1984). It is difficult to match measured hydrographs in areas with high water tables, and the various antecedent conditions (I, II, and III) cannot handle the problem accurately. The strength of the SCS method is the enormous database of soils information, soils maps, and site-specific rainfall–runoff studies. McCuen (2005) provides more details on SCS assumptions and design criteria, as compared to other operational computer models in hydrology. Example 2–6 illustrates the SCS unit hydrograph method, based on using Equation (2–18) for lag time.

SCS TRIANGULAR UNIT HYDROGRAPH

EXAMPLE 2–6

Use the SCS method to develop a UH for the area of 10 mi² described below. Use rainfall duration of $D = 2.0$ hr. Sketch the approximate shape of the triangular UH.

$$L = 5 \text{ miles}$$

$$L_c = 2 \text{ miles}$$

The watershed consists of meadows in good condition with soil group D . The average slope in the watershed is 100 ft/mi. Sketch the resulting SCS triangular hydrograph.

Equation (2–18) gives the following relationship for t_p :

SOLUTION

$$t_p = \frac{L^{0.8}(S + 1)^{0.7}}{1900\sqrt{y}}$$

From Table 2–1, the SCS curve number is found to be 78. Therefore,

$$S = 1000/\text{CN} - 10$$

$$= 1000/78 - 10,$$

$$S = 2.82 \text{ in.}$$

Converting $L = 5$ mi, or

$$L = (5 \text{ mi})(5280 \text{ ft/mi}) = 26,400 \text{ ft}$$

The slope is 100 ft/mi, so convert to percent for the equation

$$\begin{aligned} y &= (100 \text{ ft/mi})(1 \text{ mi}/5280 \text{ ft})(100) \\ &= 1.9 \end{aligned}$$

and

$$\begin{aligned} t_p &= \left[\frac{(26,400)^{0.8}(2.82 + 1)^{0.7}}{1900\sqrt{1.9}} \right] \\ &= 3.36 \text{ hr.} \end{aligned}$$

From Equation (2-17) and with rainfall duration $D = 2.0$ hr,

$$\begin{aligned} T_R &= D/2 + t_p \\ &= (2/2) + 3.36 \text{ hr,} \end{aligned}$$

$T_R = 4.36$ hr, the rise time of the hydrograph,

and Equation (2-16) gives for $A = 10 \text{ mi}^2$

$$\begin{aligned} Q_p &= \frac{484 A}{T_R} \\ &= \frac{484(10)}{4.36} \text{ cfs,} \\ [Q_p &= 1,110 \text{ cfs.}] \end{aligned}$$

To complete the graph, it is also necessary to know the time of fall B . The volume is known to be 1 in. of direct runoff over the watershed, so

$$\text{Vol} = (10 \text{ mi}^2) \left(\frac{5280 \text{ ft}}{\text{mi}} \right)^2 \left(\frac{\text{ac}}{43,560 \text{ ft}^2} \right) (1 \text{ in.}) = 6400 \text{ ac-in.}$$

From Equation (2-12),

$$\begin{aligned} \text{Vol} &= \frac{Q_p T_R}{2} + \frac{Q_p B}{2} = 6400 \text{ ac-in} = 6400 \text{ cfs-hr,} \\ 6400 \text{ cfs-hr} &= \frac{(1110 \text{ cfs} \times 4.36 \text{ hr})}{2} + \frac{(1110 \text{ cfs})(B \text{ hr})}{2}, \end{aligned}$$

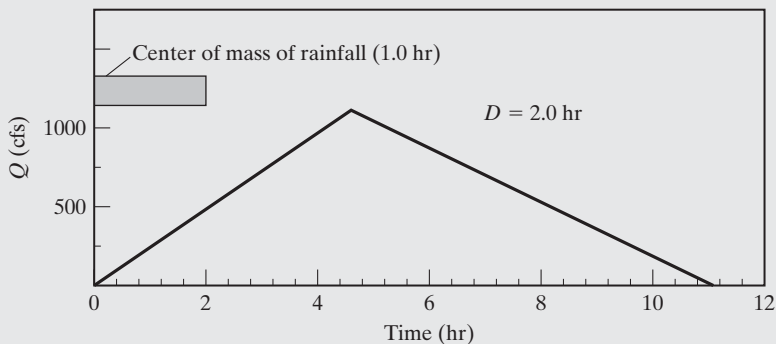
so

$$B = 7.17 \text{ hr.}$$

The triangular unit hydrograph is shown in Figure E2-6; note the time base of 11.5 hr. The next example demonstrates the use of the dimensionless SCS UH; for this example. Table E2-6 lists the resulting shaped SCS hydrograph.

Table E2-6 $T_R = 4.36$ hrs and $Q_p = 1110$ cfs

t/T_R	Q_i/Q_p	V_a/V	t (hr)	Q_i (cfs)
0	0	0	0	0
0.2	0.1	0.006	0.87	111
0.3	0.19	0.012	1.3	211
0.4	0.31	0.035	1.74	344
0.5	0.47	0.065	2.18	522
0.6	0.66	0.107	2.62	733
0.7	0.82	0.163	3.05	910
0.8	0.93	0.228	3.49	1032
0.9	0.99	0.3	3.92	1099
1.0	1.0	0.375	4.36	1110
1.2	0.93	0.522	5.23	1032
1.4	0.78	0.65	6.10	866
1.6	0.56	0.75	6.98	622
1.8	0.39	0.822	7.85	433
2.0	0.28	0.871	8.72	311
2.2	0.207	0.908	9.59	230
2.4	0.147	0.934	10.46	163
2.6	0.107	0.953	11.36	119
2.8	0.077	0.967	12.21	85
3.0	0.055	0.977	13.1	61
3.4	0.029	0.989	14.82	32
4.0	0.011	0.997	17.44	12
5.0	0	1.0	21.8	0

**Figure E2-6**
Triangular unit hydrograph for Example E2-6.

Clark UH Method

The Clark (1945) UH is based on the use of a watershed and is modeled as a linear channel in series with a linear reservoir to account for translation and attenuation, respectively. This conceptual model defines the outflow from the linear channel as inflow ($\bar{I}_i$) to the linear reservoir, and the outflow

(O_i) from the linear reservoir as the instantaneous unit hydrograph (IUH). The linear channel uses an area–time relationship (i.e., isochrones of travel time to the outlet of watershed), and it is used to estimate the time distribution of runoff from the basin. Time of concentration is represented by the time of runoff from the most remote part of the basin to the outlet. The linear reservoir signifies the combining effects of storage and resistance from the basin. Hoggan (1997) described the Clark method in details using the concept of the instantaneous unit hydrograph. Derivation starts with the relationship between the reservoir outflow and the storage is expressed as

$$S_i = RO_i, \quad (2-23)$$

where

$$\begin{aligned} S_i &= \text{storage at end of period } i, \\ O_i &= \text{outflow during period } i, \\ R &= \text{storage coefficient.} \end{aligned}$$

The continuity equation can be modified as

$$\bar{I}_i - \frac{O_{i-1} + O_i}{2} = \frac{S_i - S_{i-1}}{\Delta t}, \quad (2-24)$$

where

$$\begin{aligned} \bar{I}_i &= \text{average inflow in period } i, \text{ determined from the area–time} \\ &\quad \text{method,} \\ O_i &= \text{outflow during period } i, \\ S_i &= \text{storage at the end of period } i. \end{aligned}$$

Substituting Equation (2-23) into Equation (2-24) yields

$$\bar{I}_i - \frac{O_{i-1} + O_i}{2} = \frac{RO_i - RO_{i-1}}{\Delta t} \quad (2-25)$$

A term C is defined as in Equation (2-26):

$$C = \frac{2 \Delta t}{2R + \Delta t} \quad (2-26)$$

The outflow (O_i) from the linear reservoir can be obtained with the flow rates and duration (Δt) by combining Equations (2-25) and (2-26) as follows:

$$O_i = C\bar{I}_i + (1 - C)O_{i-1} \quad (2-27)$$

Numerical solutions of Equation (2-27) can be used to plot the instantaneous unit hydrograph (IUH).

Two major parameters of the Clark unit hydrograph are the time of concentration T_c and the storage coefficient R , which is the slope of the storage–outflow curve for the linear reservoir. T_c and R values can usually be obtained

from observed hydrographs for gaged basins: T_c is estimated as the time from the end of a burst rainfall excess to the inflection point on the receding limb; R is estimated by dividing the direct-runoff discharge at the inflection point by the slope of the curve at that point. The inflection point means where inflow to the linear reservoir is zero. At the inflection point, the continuity equation can be expressed as

$$-\frac{O_{i-1} + O_i}{2} = \frac{S_i - S_{i-1}}{\Delta t}. \quad (2-28)$$

By substituting Equation (2-23), a new equation for R is derived:

$$R = \frac{(O_{i-1} + O_i)/2}{-(O_i - O_{i-1})/\Delta t}. \quad (2-29)$$

Equation (2-29) theoretically produces the value of R as the ratio between the average discharge at the inflection point and the negative slope of the hydrograph at this point (which is negative).

For ungaged basins, T_c is usually determined from an analysis of travel time in the basin or from regression relationships that have been developed for the region. Even though the storage coefficient (R) cannot be obtained from measurable watershed characteristics directly, many regression equations have been developed from some regions. Because T_c and R are interdependent, the combining variables, $T_c + R$ and $R/(T_c + R)$ are always used in unit hydrograph computations. Thus, the Clark method is now called the TC and R method, and is routinely used in HEC-HMS models, as shown in Table 5-10.

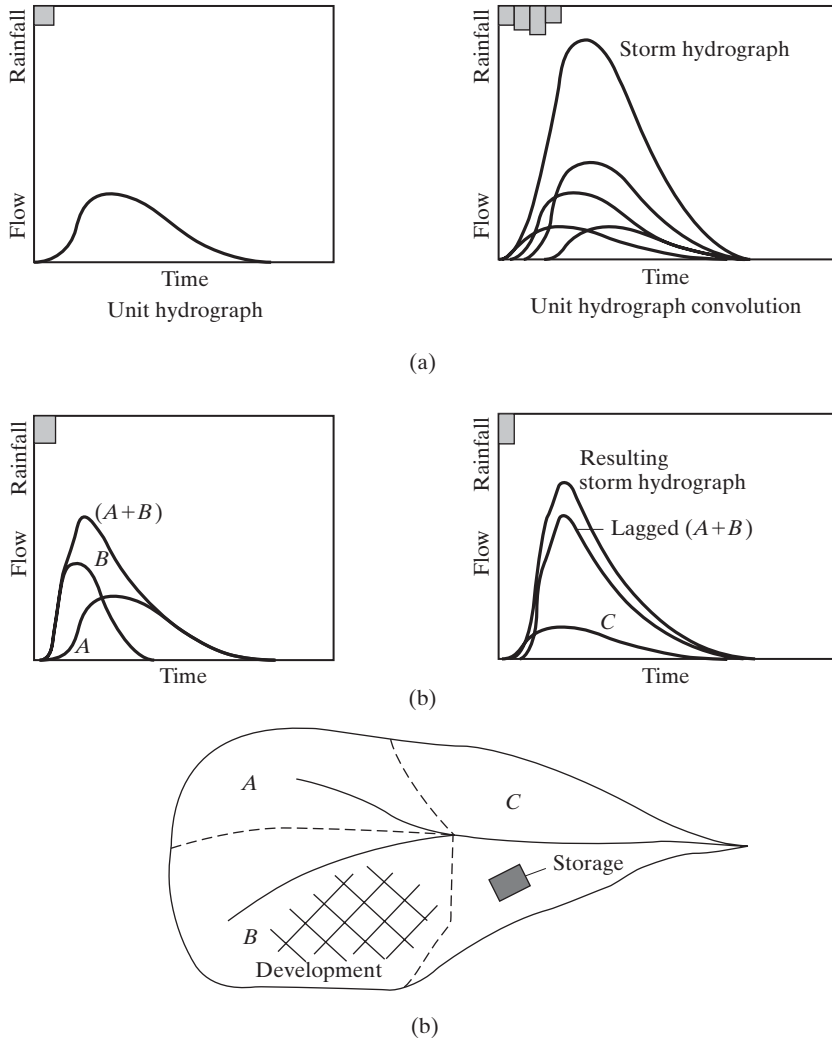
Once a UH of given duration based on known storms or synthetic methods has been developed for a particular basin under a given set of physiographic conditions, it can then be utilized for a number of hydrologic calculations (Fig. 2-9). Such hydrographs can be developed for almost any area where UH methods have been developed or by applying general methods such as SCS's or Snyder's. In particular, for a single basin area, the UH can be used with a given storm event to determine the storm hydrograph through the add-and-lag procedure. For complex watersheds of several subbasins:

1. Design storm hydrographs for selected recurrence-interval storms (10-yr, 25-yr, 100-yr) can be developed through convolution (adding and lagging) procedures for a given watershed area (Fig. 2-9(a)).
2. Effects of land use changes, channel modifications, storage additions, and other variables can be tested to determine changes in the UH (Fig. 2-9).

2.4 APPLICATIONS OF UNIT HYDROGRAPHS

Figure 2-9

Unit hydrograph applications.
 (a) Development of a design storm hydrograph.
 (b) Development of a watershed hydrograph.

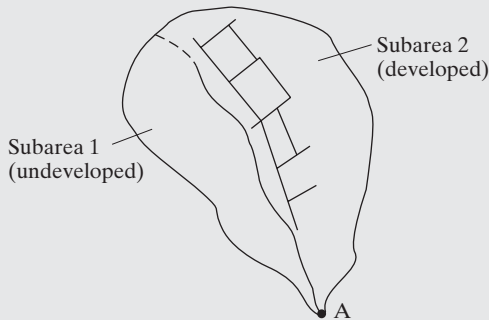


3. Storm hydrographs for each subbasin can be simulated by adding, lagging, and routing the flows produced by unit hydrographs through channel reaches (Chapter 4). Effects of various rainfall patterns and land use distributions can be tested on overall hydrologic response of the large watershed (Fig. 2-9(b)), as demonstrated in Example 2-7.
4. Storage routing methods (Chapter 4) can be used to translate inflow UHs through a reservoir or detention basin of particular size to attenuate peak flow or lag time to the peak of the hydrograph. This is used for analysis of flood control options.

HYDROGRAPH CONVOLUTION**EXAMPLE 2-7**

Given UH_1 and UH_2 for two subareas of a watershed that meet at confluence point A ; use the 10-yr 6-hr design rainfall for Houston (Fig. 1-15) to find the resulting storm hydrograph at the point of confluence. Use hydrograph convolution (add and lag). Assume infiltration losses are 0.15 in. for the first hour and 0.1 in./hr thereafter. Estimate the area of subarea 1. Figure E2-7 shows the watershed.

Time	0	1	2	3	4	5	6	7	8	9
UH_1	0	100	300	450	350	250	150	100	50	0
UH_2	0	160	480	720	560	400	240	160	80	0



Note: 1 ac-in/hr $\approx$ 1 cfs

Figure E2-7

Subareas 1 and 2. Subarea 1 is developed, while subarea 2 is undeveloped.

By definition, the area under the UH curve (volume of direct runoff) divided by the area of the watershed equals one inch of net rainfall.

Sum of UH_1 ordinates = 1750 cfs-hr = 1750 ac-in. Since this results in one inch of net rainfall, the subarea 1 is 1750 ac. (Note: 1 acre = 0.0015 sq mi)

$$1750 \text{ ac} = 2.73 \text{ sq mi.}$$

Find the 10-yr design storm volumes for the 6-hr duration and other useful durations going down to 1-hr from Figure 1-15. For a 6-yr storm, having the storm volumes for 1, 2, and 3 hr will give a clear definition of the 3 hr of highest rainfall. After that, the 6-yr storm volume will do, as there is no need to be more specific than to divide up the remaining low rainfall.

t (hr)	i (in./hr)	Rainfall (in.)
6	0.9	5.4
3	1.5	4.5
2	2	4
1	3.1	3.1

SOLUTION

Convert design rainfall to a design net hyetograph (in./hr) by placing the highest rainfall (the 1-hr value) at the center of the storm (interval 2–3 or 3–4). In this case, we will use the interval 3–4. The 2-hr value is 4.0, so 0.9 in. (4.0 in. – 3.1 in.) fell between the peak 1 hr and the next highest hour (the earlier interval of 2–3). The 3-hr value is 4.8, so 0.8 in. (4.8 in. – 3.1 in. – 0.9 in.) fell from $t = 4$ –5 hr. This is the interval directly after the peak, so now the peak of the hyetograph is defined. Divide the remaining 0.6 in. of rain over the remaining 3 hr, assuming equal intensity, and then subtract the infiltration losses each hour, which yields.

t (hr)	i (in./hr)	f (in./hr)	i_N (in./hr)
0–1	0.2	0.15	0.05
1–2	0.2	0.1	0.1
2–3	0.9	0.1	0.8
3–4	3.1	0.1	3.0
4–5	0.8	0.1	0.7
5–6	0.2	0.1	0.1
Total = 5.4 inches			

Use hydrograph convolution (Excel spreadsheet) for each subarea, and then combine the two storm hydrographs to obtain the final storm hydrograph. Subarea 1 is shown on the following page.

Time (hr)	$P_1 U_n$	$P_2 U_n$	$P_3 U_n$	$P_4 U_n$	$P_5 U_n$	$P_6 U_n$	Q (cfs)
0	0						0
1	5	0					5
2	15	10	0				25
3	23	30	80	0			133
4	18	45	240	300	0		603
5	13	35	360	900	70	0	1378
6	8	25	280	1350	210	10	1883
7	5	15	200	1050	315	30	1615
8	3	10	120	750	245	45	1173
9	0	5	80	450	175	35	745
	0	0	40	300	105	25	470
		0	0	150	70	15	235
				0	35	10	45
					0	5	5
						0	0

Subarea 2 is shown below.

Time (hr)	P_1U_n	P_2U_n	P_3U_n	P_4U_n	P_5U_n	P_6U_n	Q (cfs)
0	0						0
1	8	0					8
2	24	16	0				40
3	36	48	128	0			212
4	28	72	384	480	0		964
5	20	56	576	1440	112	0	2204
6	12	40	448	2160	336	16	3012
7	8	24	320	1680	504	48	2584
8	4	16	192	1200	392	72	1876
9	0	8	128	720	280	56	1192
		0	64	480	168	40	752
			0	240	112	24	376
				0	56	16	72
					0	8	8
						0	0

The resulting storm flow is calculated by adding the flows together from subareas 1 and 2. Final peak flow is 4895 cfs at hour 6, as shown below in the shaded column.

Time (hr)	Q_1 (cfs)	Q_2 (cfs)	Total Q (cfs)
0	0	0	0
1	5	8	13
2	25	40	65
3	133	212	345
4	603	964	1567
5	1378	2204	3582
6	1883	3012	4895
7	1615	2584	4199
8	1173	1876	3049
9	745	1192	1937
10	470	752	1222
11	235	376	611
12	45	72	117
13	5	8	13
14	0	0	0

The above computations show the final peak of 4895 cfs at hour 6. They can be easily done in Excel spreadsheets, but for more complex watersheds and more complex rainfalls, where flood routing in streams might be needed, it is usually necessary to use computer models such as those presented in Chapters 5 and 6.

2.5 LINEAR AND KINEMATIC WAVE MODELS

Instantaneous Unit Hydrographs

Nash suggested that the two-parameter gamma distribution gave the general shape of an **instantaneous unit hydrograph** (IUH), produced from a unit rainfall falling for time D as D approaches zero. The IUH was shown to be equivalent to the output from a cascade of linear reservoirs. (A linear reservoir is a reservoir with outflow that is linearly proportional to the storage volume.) Gray (1962) later based a popular UH method on the same gamma distribution (see Chapter 3).

A useful mathematical extension of UHs of finite duration can be developed if the duration of rainfall excess D approaches zero while the quantity (unit depth) remains constant. The runoff produced by this “instantaneous rainfall” is called the **instantaneous unit hydrograph**. The IUH is a response function for a particular watershed to a unit impulse of rainfall excess. By summing up the outputs due to all instantaneous inputs through time, one arrives at $Q(t)$, the output hydrograph.

The IUH is assumed to be a unique function for a watershed, independent of time or antecedent conditions. The output function $Q(t)$, or total storm discharge, is produced by summing the outputs due to all instantaneous inputs $i(t)$. If the input is a succession of inputs of volume $i(\tau)d\tau$, then each adds its contribution $i(\tau)u(t - \tau)d\tau$ to the rate of output Q at time t . Stated mathematically, the runoff rate at any fixed time t is

$$Q(t) = \int_0^t i(\tau)u(t - \tau) d\tau, \quad (2-30)$$

where $i(\tau)$ is the rainfall excess at time τ (Fig. 2-10) and $u(t - \tau)$ can be viewed as a weighting function given to rainfall intensities that occurred at time $(t - \tau)$ before. The integral, known as the convolution integral, gives the output runoff as a continuous function of time t . Equation (2-1) from Section 2.2 is the discrete form of Equation (2-30).

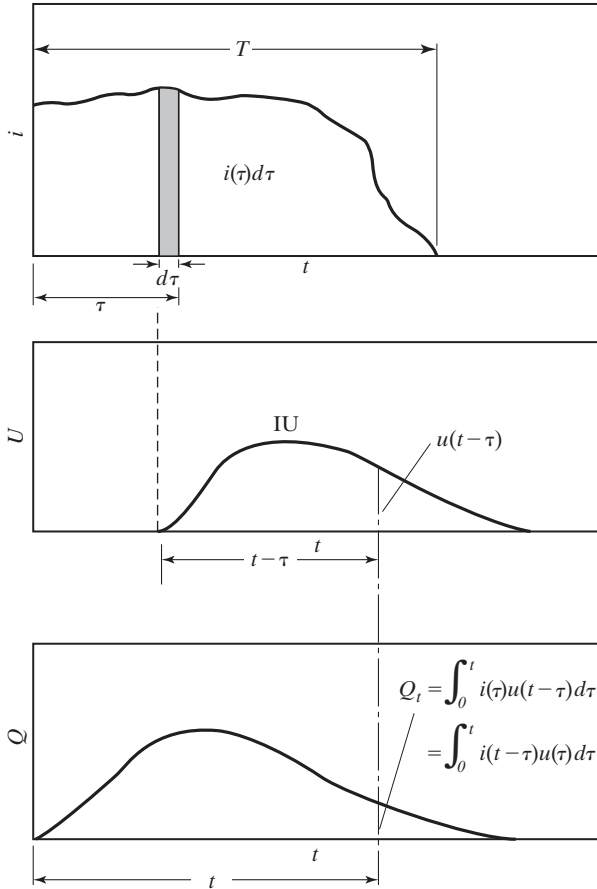
The real usefulness of the IUH as a mathematical concept can be seen in relation to the S-curve presented in Section 2.2. Consider an S-curve formed by a continuous rainfall of unit intensity, which can be an infinite series of very small time units, each separated by τ . Summing all the individual UHs of duration τ , the S-curve ordinates become

$$S(t) = \tau[u(t) + u(t - \tau) + u(t - 2\tau) + \cdots] \quad (2-31)$$

since τ also equals the depth for a unit intensity. Then, as $\tau \rightarrow 0$ in the limit for the IUH,

$$S(t) = \int_0^t u(t)dt. \quad (2-32)$$

Therefore, the S-curve is the integral of the IUH, and the IUH is the first derivative of the S-curve. The slope of the S-curve, dS/dt , is proportional to the ordinate of the IUH.

**Figure 2-10**

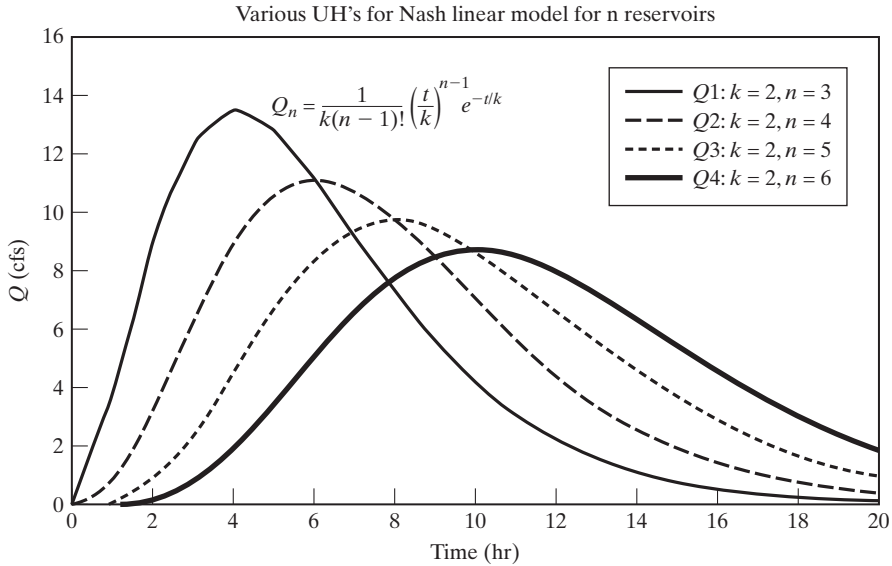
Graphical representation of how to use an IUH to generate a hydrograph.

The IUH has been widely used, although it suffers the same disadvantage as the UH in that rainfall-runoff responses may be nonlinear and dependent on antecedent conditions. Nonlinear models for the unit hydrograph have been developed in the general literature and are reviewed by Chow (1964) and Raudkivi (1979).

Nash (1959) presented a linear UH model based on n reservoirs in series to represent a watershed, each with a linear storage relation $S = KQ$, that resulted in a gamma distribution for shape. Figure 2-11 shows various shapes that result for selecting two parameters n and K in the model.

Kinematic Wave Methods for Overland Flow

Henderson and Wooding (1964) and Wooding (1965) developed a unique theory for overland flow and the stream hydrograph based on the concept of the **kinematic wave**, which assumes that the weight or gravity force of flowing

**Figure 2-11**

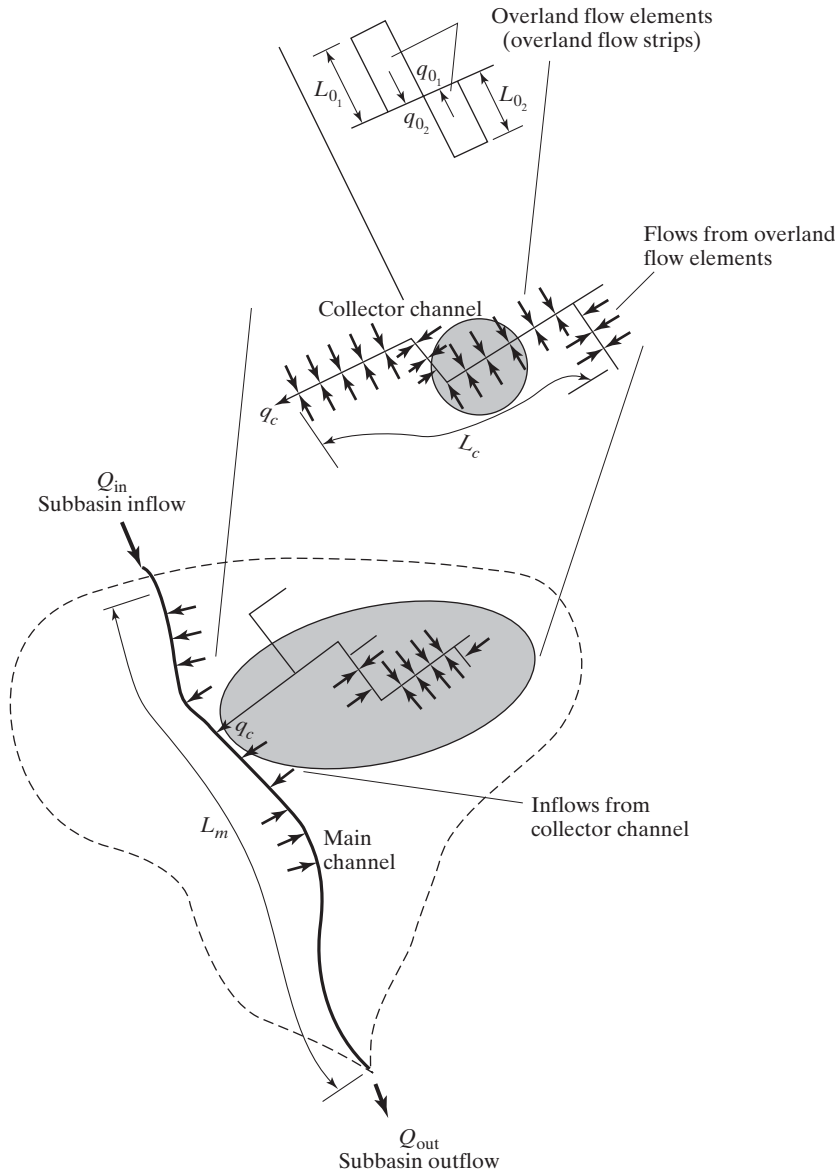
Various UHs based on the Nash linear model.

water is simply balanced by the resistive forces of bed friction. All flows are assumed to obey the equations of continuity and momentum (e.g., Manning's equation) as shown in Section 4.6. A complete derivation of the concept of the kinematic wave is presented there, and it is mentioned here because the kinematic wave method is a physics-based approach similar to the unit hydrograph for watershed areas. Modern distributed models solve these equations numerically over a finite grid that is used to represent the variation of elevation and channel configuration for a basin area (Chapter 11).

Solution methods to above equations for the kinematic wave using the method of characteristics are described in detail by Lighthill and Whitham (1955), Eagleson (1970), Overton and Meadows (1976), Raudkivi (1979), Stephenson and Meadows (1986), and Singh (1996). Most practical applications of kinematic wave methods require the use of numerical methods because of nonuniform rainfall and variable basin characteristics and are presented in Chapter 4 along with other numerical flood routing procedures. Modern distributed models are based on these equations, and an example model and application are presented in Chapter 11.

Kinematic (KW) wave routing can be used to derive overland flow hydrographs, which can be added to produce collector or channel hydrographs and, eventually, can be flood routed as channel or stream hydrographs. In the early 1980s, the kinematic wave method was formally added to several available computer models, such as the HEC-HMS from the U.S. Army Corps of Engineers (Hydrologic Engineering Center, 2010). Figure 2-12

Figure 2-12
Conceptual model of
the kinematic wave.



shows the kinematic wave concept, which uses a number of interconnected elements, including overland flow elements, collector channels or pipes, and main channels, to describe an overall watershed. Explicit numerical methods (Section 4.6) are employed to solve equations for each element, and overland flow becomes input to the collector system, which eventually forms the lateral input hydrograph to a main channel. Chapter 4 presents kinematic

wave flood routing in detail, and Chapter 5 presents some applications for the HEC-HMS models. The KW method also is a main driver of the Storm Water Management Model, described in Chapter 6, and the distributed model VFlo, presented in Chapter 11. Stephenson and Meadows (1986), Singh (1996), and Vieux (2002) provide excellent reviews of kinematic wave modeling methods.

2.6
HYDROLOGIC
LOSS—
EVAPORATION
AND ET

Introduction to Evaporation

Evaporation is the process by which water in its liquid or solid state is transformed into water vapor, which as a gas mixes with the atmosphere. Evapotranspiration (*ET*) is considered separately as the combined loss of water vapor from the surface of plants (transpiration) and the evaporation of moisture from soil. Knowledge of evaporation processes is important in predicting water losses to evaporation from a lake or reservoir. On average, approximately 70% of the mean annual rainfall in the United States is returned to the atmosphere as evaporation or transpiration. However, variations in evaporation across the continent can be very large due to effects of solar input, location of mountains, and proximity to oceans. On an annual basis, evaporation rates can exceed mean annual rainfall, especially in arid regions of the southwest. For example, arid areas in Texas, Arizona, New Mexico, Nevada, and southern California can exceed 70 in. per year, compared to about 30 to 40 in. per year for much of the rest of the country.

For the case of evaporation from a lake surface, water loss is a function of solar radiation, temperature of the water and air, difference in vapor pressure between water and the overlying air, and wind speed across the lake. As evaporation proceeds in a closed-container system at a constant temperature, pressure in the air space increases because of an increase in partial pressure of water vapor. Evaporation continues until vapor pressure of the overlying air equals the surface vapor pressure; at this point, the air space is said to be saturated at that temperature, and further evaporation ceases. This state of equilibrium would not be reached if the container were open to the atmosphere, in which case all water would eventually evaporate. Thermal energy is required to increase the free energy of water molecules to allow escape across the liquid–gas interface. The amount of heat required to convert water to vapor at constant temperature (597 cal/g at 0°C) is called the latent heat of evaporation, as given in Equation (1–5).

As vaporization continues over a flat free-water surface, an accumulation of vapor molecules causes an increase in the vapor pressure e in the air just above the water surface, until eventually condensation begins. The air is saturated when the rate of condensation equals the rate of vaporization and e equals the saturation vapor pressure. However, various convective transport processes operate to transport the vapor (by wind-driven currents) and prevent equilibrium from occurring.

Evaporation is important in the long-term water balance and is usually of concern for large-scale water resources planning and water supply studies. During typical storm periods, with intensities of 0.5 in./hr, evaporation is on the order of 0.01 in./hr and is normally neglected for flood flow studies and urban drainage design applications. Evaporation has been extensively studied in the United States since the 1950s, beginning with the comprehensive Lake Hefner evaporation research project by Marciano and Harbeck (1954). Three primary methods are used to estimate evaporation from a lake surface: the water budget method, the mass transfer method, and the energy budget method. These are discussed in more detail in the sections that follow, and Brutsaert (1982) and Dingman (2002) present a more detailed review of evaporation mechanisms.

Water Budget Method for Determining Evaporation

The water budget method for lake evaporation is based on the continuity equation. Assuming that change in storage ΔS , surface inflow I , surface outflow O , subsurface seepage to ground water flow GW , and precipitation P can be measured, evaporation E can be computed as

$$E = -\Delta S + I + P - O - GW. \quad (2-33)$$

The approach is simple in theory, but evaluating seepage terms can make the method quite difficult to implement. The obvious problems result from errors in measuring precipitation, inflow, outflow, change in storage, and subsurface seepage. Good estimates using the method were obtained for Lake Hefner, near Oklahoma City, with 5% to 10% error. It should be emphasized that Lake Hefner was selected from more than one hundred lakes and reservoirs as one of three or four that best met water budget requirements and had minimal seepage losses to the subsurface.

Mass Transfer Method

Mass transfer techniques are based primarily on the concept of turbulent transfer of water vapor from a water surface to the atmosphere. Numerous empirical formulas have been derived to express evaporation rate as a function of vapor-pressure differences and wind speed above a lake or reservoir. Many such equations can be written in the form of a diffusive flux of water vapor, whose concept dates back to Dalton's original theories (Brutsaert, 1982):

$$E = (e_s - e_a)(a + bu), \quad (2-34)$$

where

- e_s = saturation vapor pressure at the T_s of the water surface,
- e_a = vapor pressure at some fixed level above the water surface, the product of relative humidity times saturation vapor pressure at T_a of the air,
- u = wind speed,
- a, b = empirical constants.

An obstacle to comparing different evaporation formulas is the variability in measurement heights for u and e_a . Reducing all formulas to the same measurement level of 2 m (6.5 ft) for wind speed and vapor pressure and taking into account the 30% difference between reservoir and pan evaporation, the scatter between the common formulas is considerably reduced. The formula with the best database is the Lake Hefner formula given by Marciano and Harbeck (1954), which also performed well at Lake Mead (Harbeck, 1958). Some formulas use a zero value of the constant a in Equation (2-34), due probably to small local air movements with velocities insufficient to remove excess vapor from above a pan surface. Harbeck and Meyers (1970) present the formula

$$E = bu_2(e_s - e_2), \quad (2-35)$$

where

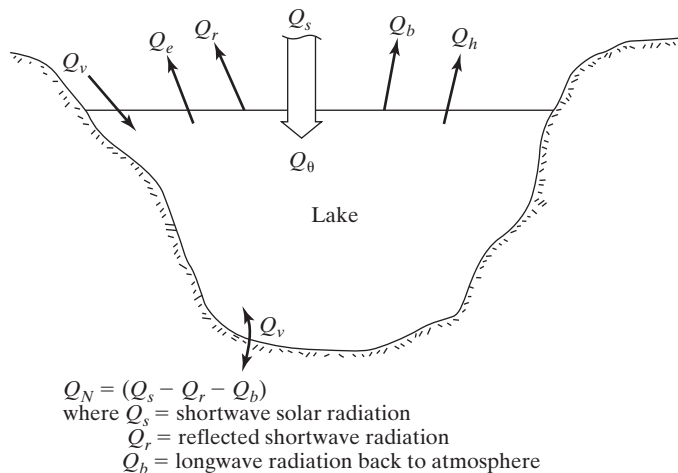
- E = evaporation (cm/day),
- b = 0.012 for Lake Hefner, 0.0118 for Lake Mead
($\text{cm-day}^{-1}\text{-m}^{-1}\text{-s-mb}^{-1}$),
- e_s = vapor pressure at water surface (mb),
- e_2 = vapor pressure 2 m above water surface (mb),
- u_2 = wind speed 2 m above the water surface (m/s).

Energy Budget Method

The most accurate and complex method for determining evaporation uses the energy budget of a lake (Fig. 2-13). The overall energy budget for a lake can be written in langley/day, where 1 langley (Ly) = 1 cal/cm^2 :

$$Q_N - Q_h - Q_e = Q_\theta - Q_v, \quad (2-36)$$

Figure 2-13
A lake's energy budget.



where

- Q_N = net radiation absorbed by the water body,
- Q_h = sensible heat transfer (conduction and convection to the atmosphere),
- Q_e = energy used for evaporation,
- Q_θ = increase in energy stored in the water body,
- Q_v = advected energy of inflow and outflow.

Letting L_e represent the latent heat of vaporization (cal/g) and R the ratio of heat loss by conduction to heat loss by evaporation, Equation (2-36) becomes

$$E = \frac{Q_N + Q_v - Q_\theta}{\rho L_e(1 + R)}, \quad (2-37)$$

where E is the evaporation (cm/day) and ρ is the density of water (g/cm³). The Bowen ratio (R) is used as a measure of sensible heat transfer and can be computed by

$$R = 0.66 \left(\frac{T_s - T_a}{e_s - e_a} \right) \left(\frac{P}{1000} \right) = \gamma \frac{T_s - T_a}{e_s - e_a}, \quad (2-38)$$

where

- P = atmospheric pressure (mb),
- T_a = air temperature (°C),
- T_s = water surface temperature (°C),
- e_a = vapor pressure of the air (mb),
- e_s = saturation vapor pressure at surface water temperature (mb),
- γ = the psychrometric constant $0.66P/1000$ (mb/°C).

Application of the energy budget method requires measurements of total incoming or net radiation. The Bowen ratio was conceived because sensible heat transfer cannot be readily computed. The method was applied to Lake Hefner and Lake Mead and was used to evaluate empirical coefficients for the mass transfer method and to interpret pan evaporation data at Lake Hefner. The energy budget method is theoretically the most accurate, but it requires the collection of large amounts of detailed atmospheric data, which are sometimes not available. To get around this problem, other methods such as pan evaporation methods have been developed to estimate shallow-lake evaporation, which is used at many stations across the United States.

Pan Evaporation

Evaporation can be measured from a standard Weather Bureau class A pan, an open galvanized iron tank 4 ft in diameter and 10 in. deep mounted 12 in. above the ground (see Fig. 1-29). To estimate evaporation, the pan is filled to a depth of 8 in. and must be refilled when the depth has fallen to 7 in. The water surface level is measured daily, and evaporation is computed as the

difference between observed levels, adjusted for any precipitation measured in a standard rain gage. Alternatively, water is added each day to bring the level up to a fixed point. Pan evaporation rates are higher than actual lake evaporation and must be adjusted to account for radiation and heat-exchange effects. The adjustment factor is called the pan coefficient, which ranges from 0.64 to 0.81 and averages 0.70 for the United States. However, the pan coefficient varies with season, exposure, and climatic conditions and should be used only for rough estimates of lake evaporation. Pan evaporation data are archived by the National Climatic Data Center and are available at a number of stations in the United States (Farnsworth and Thompson, 1982; Dingman, 2002).

Combined Methods

Penman (1948) first used the best features of the mass transfer and energy budget methods to derive a water surface evaporation relation that is fairly easy to compute. The Penman equation is

$$E_h = \frac{\Delta}{\Delta + \gamma} Q_N + \frac{\gamma}{\Delta + \gamma} E_a, \quad (2-39)$$

where

E_h = flux of latent heat due to evaporation (energy/area-time)
= $\rho L_e E$ with E in units of L/T ,

L_e = latent heat of vaporization [Eq. (1-5)], customarily evaluated at the temperature of the air (energy/mass),

Δ = slope of e_s -vs.- T curve, which is shown as Δ/γ vs. T in Van Bavel (1966).

It is customary to evaluate Δ/γ or just Δ at the temperature of the air, not the temperature of the water surface. Alternatively, since Δ represents the slope of saturated vapor pressure vs. temperature, it can be obtained by differentiating Equation (1-4), thus

$$\Delta = \frac{de_s}{dT} = \frac{2.7489 \times 10^8 \times 4278.6}{(T + 242.79)^2} \exp\left(-\frac{4278.6}{T + 242.79}\right) \quad (2-40)$$

with units for Δ of $\text{mb}/^\circ\text{C}$, and T in $^\circ\text{C}$.

Q_N = net radiation absorbed in Equation (2-36) (energy/area-time)

γ = psychrometric constant from Equation (2-38) ($\text{mb}/^\circ\text{C}$)

E_a represents the “drying power” of the air (Brutsaert, 1982) and is given by

$$E_a = \rho L_e (a + bu)(e_{sa} - e_a), \quad (2-41)$$

where

E_a has units of energy flux (energy/area-time),

$a + bu$ = empirical transfer constants from Equation (2-34) or (2-35) (L/T) per unit of pressure,

e_{sa} = saturation vapor pressure at temperature of the air,
 e_a = actual vapor pressure in air $RH \approx e_{sa}$,
 RH = relative humidity (fraction).

The Penman equation has the advantage that the water or soil surface temperature need not be known. It has thus been found very useful for evapotranspiration studies, in which it is difficult to determine the surface temperature of vegetation (Jensen et al., 1990). When the surface temperature of the water can be readily measured, the energy budget–Bowen ratio approach discussed previously is probably better because it avoids the need to use an empirical transfer coefficient ($a + bu$).

The Penman equation is the basis for the graphical regression procedure shown in Figure 2–14 (Kohler et al., 1955); this regression is illustrated in Example 2–8. The use of the Penman equation itself [Eq. (2–39)] is shown in Example 2–9.

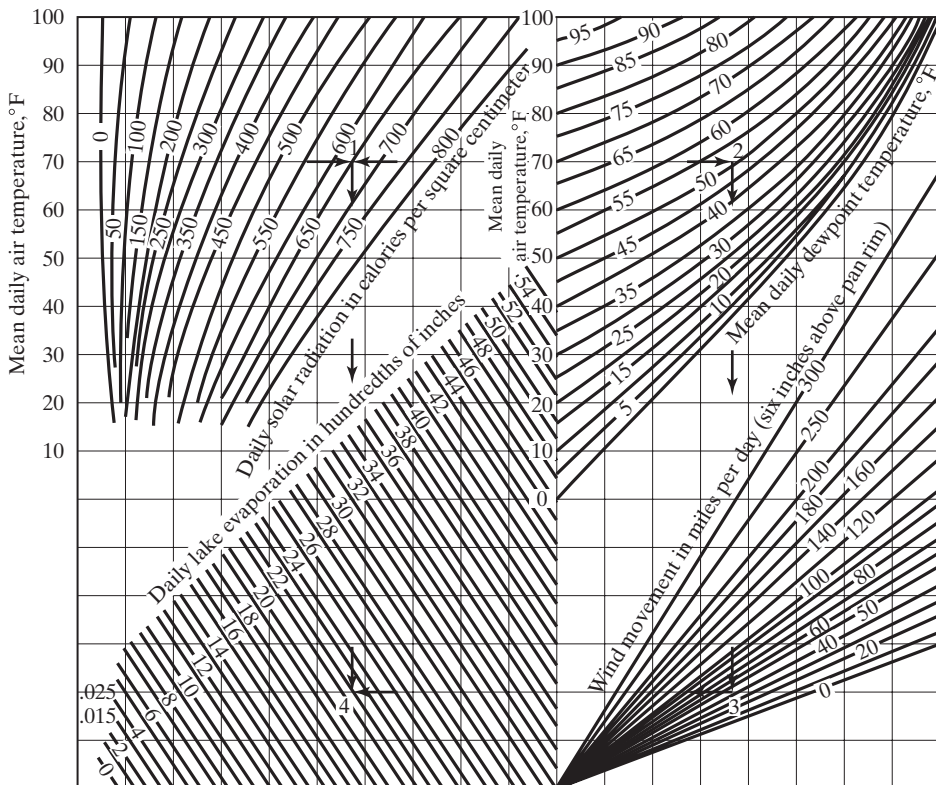


Figure 2–14

Shallow lake evaporation as a function of solar radiation, air temperature, dew point, and wind movement (Adapted from Kohler et al., 1995).

EXAMPLE 2-8**DETERMINING EVAPORATION USING GRAPHICAL REGRESSION**

For a given shallow lake, the mean daily air temperature T_a is 70°F. The solar radiation is measured as 650 ly/day. The mean daily dew point temperature is 50°F, and the wind movement at 6 in. above the pan rim is 40 mi/day. Using the nomograph for shallow-lake evaporation shown in Figure 2-14, determine the daily lake evaporation in inches.

SOLUTION

Point 1 on Figure 2-14 shows the point where $T_a = 70^\circ\text{F}$ and the solar radiation is 650 ly/day. Point 2 is drawn at the intersection of the curve of $T_s = 50^\circ\text{F}$ and $T_a = 70^\circ\text{F}$. A vertical line is drawn from point 2 to the curve representing wind movement equal to 40 mi/day. This is point 3. Point 4 is found at the intersection of a horizontal line projected from point 3 and a vertical line projected from point 1. The daily lake evaporation is then read from the graph as

$$E = 0.22 \text{ in.}$$

EXAMPLE 2-9**EVAPORATION USING THE PENMAN EQUATION**

An example of Equation (2-34) is given by Meyer (1944) for Minnesota lakes:

$$E = 0.0106(1 + 0.1u)(e_s - e_a)$$

with E in in./day, u in mph, and vapor pressures in mb. For an air temperature of 90°F (32.2°C), wind speed of 20 mph, relative humidity of 30%, and net radiation flux of 400 ly/day, estimate the evaporation rate using the Penman equation. Assume atmospheric pressure ≈ 1000 mb, so the psychrometric constant $\gamma = 0.66 \text{ mb}/^\circ\text{C}$.

SOLUTION

We evaluate Δ from Equation (2-40) at the air temperature of 32.2°C:

$$\begin{aligned}\Delta &= \frac{de_s}{dT} = \frac{2.7489 \times 10^8 \times 4278.6}{(32.2 + 242.79)^2} \exp\left(\frac{4278.6}{32.2 + 242.79}\right) \\ &= 2.72 \text{ mb}/^\circ\text{C}.\end{aligned}$$

For Equation (2-39), we need the saturation vapor pressure at 32.2°C. From Equation (1-6), we then have

$$e_{sa} = 2.7489 \times 10^8 \exp[-4278.6/(32.2 + 242.79)] = 48.1 \text{ mb.}$$

Thus, $e_a = e_{sa} = 0.3(48.1) = 14.4 \text{ mb}$. The latent heat of vaporization at the air temperature of 32.2°C is

$$L_e = 597.3 - 0.57(32.2) = 579 \text{ cal/g.}$$

The density of water will be taken as 1 g/cm^3 . We include a change in units from in./day to cm/day while evaluating E_a ,

$$\begin{aligned} E_a &= 0.0106(1 + 0.1 \cdot 20)(\text{in./day} \cdot \text{mb}) 2.54 (\text{cm/in.}) 1 (\text{g/cm}^3) \\ &\quad \times 579(\text{cal/g})(48.1 - 14.4)(\text{mb}) \\ &= 1590 \text{ cal/cm}^2\text{-day} = 15901 \text{ y/day.} \end{aligned}$$

The Penman evaporation energy flux is thus (Eq. 2-39)

$$\begin{aligned} E_h &= 2.72/(2.72 + 0.66) \cdot 400 + 0.66/(2.72 + 0.66) \cdot 1590 \\ &= 632 \text{ ly/day.} \end{aligned}$$

This can be converted to depth/day by dividing by ρL_e :

$$E = 632/1(579) = 1.09 \text{ cm/day} = 0.43 \text{ in./day of evaporation.}$$

Evapotranspiration

For a water budget over the whole watershed, one is usually concerned with the total evaporation from all free-water surfaces, plus **transpiration**, the loss of vapor through small openings (stomata) in plant tissue (e.g., leaves). For most plants, transpiration occurs only during daylight hours during photosynthesis, which can lead to diurnal variations in the shallow ground water table in heavily vegetated areas. The combined evaporation and transpiration loss is called **evapotranspiration (ET)** and is a maximum if the water supply to both the plant and soil surface is unlimited. The maximum possible loss is limited by meteorological conditions and is called **potential ET** (Thornthwaite, 1948); potential ET is approximately equal to the evaporation from a large, free-water surface such as a lake. Thus, methods discussed in the previous section for evaporation can be used to predict potential ET.

Large surface area and high temperature of leaves can easily create transpiration rates that equal or even exceed potential ET. Exceedance is possible in instances of an “oasis” of well-watered vegetation, such as a crop or golf course, located in a larger area in which actual ET is less than the potential. Actual transpiration and, hence, actual ET are usually limited by moisture supply to the plants. Obviously the effect of limited moisture will depend on plant characteristics such as root depth and the ability of the soil to transport water to the roots. These effects have led to various empirical factors that may be applied to potential ET, computed using one of the evaporation methods discussed previously. For example, Penman (1950) determined that ET from a vegetated land surface in Great Britain was 60% to 80% of potential ET computed using his method; Shih et al. (1983) used a value of 70% for crops in southern Florida. On the other hand, Priestly and Taylor (1972) multiplied Penman equation evaporation by 1.26 for well-watered crops to compute ET. Other empirical formulas

Table 2–2 Pan Coefficients for Evapotranspiration Estimates

Type of Cover	Pan Coefficient	Reference
St. Augustine grass	0.77	Weaver and Stephens (1963)
Bell peppers	0.85–1.04	
Grass and clover	0.80	
Oak–pine flatwoods (east Texas)	1.20	Englund (1977)
Well-watered grass turf		Shih et al. (1983)
Light wind, high relative humidity	0.85	
Strong wind, low relative humidity	0.35	
Everglades agricultural areas	0.65	
Irrigated grass pasture (central California)	0.76	Hargreaves and Samani (1982)

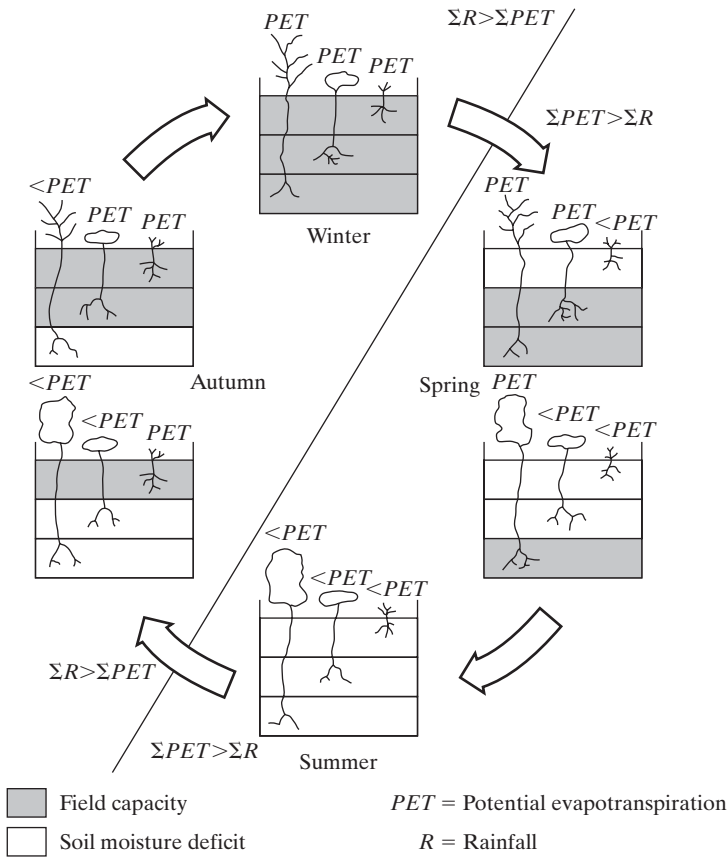
that incorporate radiation, air temperature, and precipitation are given by Thornthwaite (1948), Turc (1954), and Hamon (1961), and in agricultural practice the Blaney–Criddle method is widely used (Blaney and Criddle, 1950; Criddle, 1958).

Once again, pan evaporation can be used to estimate ET if coefficients are known for the specific vegetation. There is a wide range of pan coefficient values, as indicated in Table 2–2. Caution should be used with any pan coefficient method for prediction of ET, since the method is weak to begin with (Brutsaert, 1982) and coefficients will experience a strong seasonal variation that follows vegetative growth patterns.

The complex annual relationship among potential ET, rainfall, and soil moisture is illustrated in Figure 2–15 for plants with three different root depths. The **field capacity** of the soil is the moisture content above which water will be drained by gravity. The **wilting point** of the soil is the moisture content below which plants cannot extract further water. As the soil moisture content is reduced below field capacity, actual ET becomes less than potential ET. If the soil water content is reduced below the wilting point, the plant may die. This complicated process depends on the plant, soil type, meteorology, and season. A full discussion is beyond the scope of this text; further details may be found in Brutsaert (1982), Hillel (1982), Salisbury and Ross (1969), ASCE (2005), and Dingman (2002).

Tabulated Evapotranspiration Estimates

Most ET estimates today are based on a modification of the Penman equation for specific assumptions about transfer coefficients [Eq. (2–41)] and vegetative surface. The Penman–Monteith equation (ASCE, 2005) is one example, in which the Penman equation is applied to different crops on the basis of their roughness, height, and stomatal resistance to transpiration. The Kimberly–Penman equation, developed by the U.S. Department of Agriculture at Kimberly, Idaho, was developed using data to refine transfer coefficients

**Figure 2-15**

Idealized annual soil moisture cycle for three vegetation types.

for reference crops of alfalfa and well-clipped fescue grass (ASCE, 2005). ASCE (2005) recommends a standardized form of such an equation as the basis for ET estimates from many different vegetative surfaces. The methods work by computing a reference ET estimate based on local meteorology, and then multiplying by crop coefficients that incorporate crop-specific stomatal resistance and surface roughness to convert the reference estimates to ET estimates for each crop. The crop coefficients vary by location, season, and growth stage of the crop. Reference ET is updated daily on the basis of local meteorology.

An example of dissemination of these very useful data (e.g., for irrigation estimates) is the AgriMet service of the Bureau of Reclamation in the U.S. Pacific Northwest and northern California and Nevada (<http://www.usbr.gov/pn/agrimet/general.html>). Daily reference ET estimates are computed using the Kimberly–Penman equation for a reference crop of alfalfa for more than 70 weather stations in the region. Crop coefficients are site specific and available for almost all crops grown in a locality. Real-time and

historic weather data may also be extracted from the weather stations, making AgriMet a valuable data resource for more than just ET estimates. Application of reference ET toward ET estimates for natural vegetation (e.g., forests, grasslands) depends not only on obvious location and vegetation-specific parameters but also on available moisture content. However, such estimates can be attempted with additional information, as explained by ASCE (1996). The advantage of applying reference ET estimates to natural watersheds is the use of real-time, site-specific weather data for the reference ET computation, which are much more available than are pan measurements.

Interception

Some rainfall on vegetation is trapped on leaves and other vegetative surfaces and subsequently lost to the atmosphere through evaporation. The trapped water, or interception, does not reach the ground surface and is a loss of water that might otherwise potentially run off from the catchment. Rainfall that drops through the vegetation is known as through-fall. Interception is an example of depression storage or initial abstraction that is subtracted from rainfall during the initial stage of a storm (Chapters 2 and 6). Because the ultimate fate of such water is evaporation, interception is sometimes lumped with overall basin ET. However, estimates are available as a function of plant type, location, and season. Clearly, leafy deciduous trees may intercept considerable water in the summer and almost none in the winter. On an annual basis, Dingman (2002) reports annual interception as a percent of annual precipitation on the order of 24%–35% for Pacific Northwest conifers and on the order of 10% for North Carolina hardwoods. On a storm event basis, Shuttleworth (1993) reports canopy interception on the order of 1.2 mm for coniferous forests and 0.8 mm for broadleaf canopies in full leaf. Interception capacity is recovered during interevent dry periods as a function of evaporation rates.

2.7 HYDROLOGIC LOSS— INFILTRATION

The process of infiltration has been widely studied and represents an important mechanism for the movement of water into the soil under gravity and capillarity forces. Infiltration volume is subtracted from a precipitation event in order to determine the net volume of rainfall, or rainfall excess, which is equivalent to the direct runoff from a watershed area. Horton (1933) showed that when the rainfall rate i exceeds the infiltration rate f , water infiltrates the surface soils at a rate that generally decreases with time. These rates are usually reported in inches per hour. For any given soil, a limiting curve defines the maximum possible rates of infiltration vs. time. The rate of infiltration depends in a complex way on rainfall intensity, soil type, surface condition, and vegetal cover.

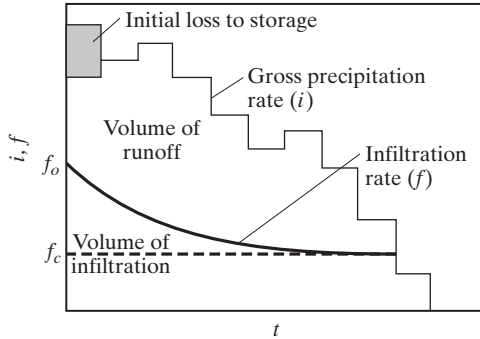


Figure 2-16
Horton's infiltration
concept.

For excess rates of rainfall, the actual infiltration rate will follow the limiting curve shown in Figure 2-16, called the **Horton infiltration capacity** curve of the soil. The capacity decreases with time and ultimately reaches a constant rate, caused by filling of soil pores with water, which reduces capillary suction. For example, it has been shown through controlled tests that the decline is more rapid and the final constant rate is much lower for clay soils than for sandy soils. Thus, sand will generally infiltrate more volume at a higher rate than clay soils.

Simple Equations for Infiltration Rate

The hydrologic concept of infiltration capacity is empirically based on observations at the ground surface. Horton (1940) suggested the following form of the infiltration equation, where rainfall intensity $i > f$ at all times:

$$f = f_c + (f_0 - f_c)e^{-kt}, \quad (2-42)$$

where

- f = infiltration capacity (in./hr),
- f_0 = initial infiltration capacity (in./hr),
- f_c = final capacity (in./hr),
- k = empirical constant (hr^{-1}).

Rubin and coworkers (1963, 1964) showed that Horton's observed curves can be theoretically predicted, given the rainfall intensity, the initial soil moisture conditions, and a set of unsaturated characteristic curves for the soil. They showed that the final infiltration rate is numerically equivalent to the saturated hydraulic conductivity of the soil. Furthermore, Rubin showed that ponding at the surface will occur if rainfall duration is greater than the time required for soil to become saturated at the surface. Horton's equation is depicted graphically in Figure 2-16, and Example 2-10 illustrates its use.

EXAMPLE 2-10**HORTON'S INFILTRATION EQUATION**

The initial infiltration capacity f_0 of a watershed is estimated as 1.5 in./hr, and the time constant is taken to be 0.35 hr^{-1} . The equilibrium capacity f_c is 0.2 in./hr. Use Horton's equation to find (a) the values of f at $t = 10 \text{ min}$, 30 min, 1 hr, 2 hr, and 6 hr, and (b) the total volume of infiltration over the 6-hr period.

SOLUTION

Horton's Equation [Eq. (2-42)] is

$$f = f_c + (f_0 - f_c)e^{-kt}$$

Substituting the values for f_0 , f_c , and k gives

$$f = 0.2 \text{ in./hr} + 1.3(e^{-0.35t}) \text{ in./hr.}$$

Solving for each value of t gives the following table:

$t \text{ (hr)}$	$f \text{ (in./hr)}$
1/6	1.43
1/2	1.29
1	1.12
2	0.85
6	0.36

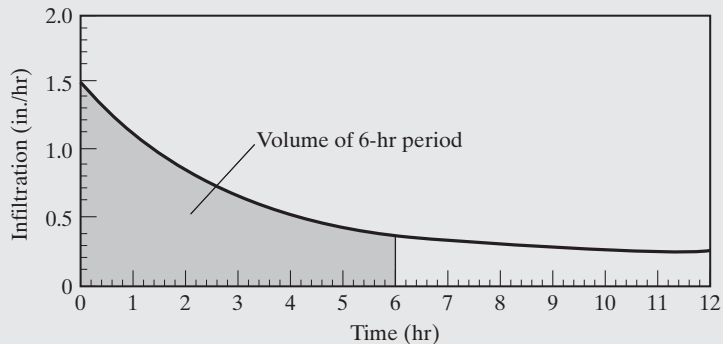
The volume (in inches over the watershed) can be found by plotting the curve given by the table of values and then finding the area under the curve bounded by $t = 0$ and $t = 6 \text{ hr}$. The plot is shown in Figure E2-10. The curve is given by the equation below, and to find F , the volume of infiltration and the area under the F curve, we must integrate Horton's equation.

$$F' = f_c + \frac{(f_0 - f_c)e^{-kt}}{-k}$$

$$f = 0.2 \text{ in./hr} + 1.3(e^{-0.35t}) \text{ in./hr.}$$

Figure E2-10

Horton infiltration curve. The area shaded under the curve represents the total volume of infiltration over a 6-hr period.



Integrating over the interval $t = 0$ to $t = 6$ hr gives

$$\begin{aligned}\text{Vol} &= \int f dt \\ &= \int 0.2 + 1.3(e^{-0.35t}) dt \\ &= [0.2t + (1.3 - 0.35)e^{-0.35t}]_0^6 \\ &= 4.46 \text{ in. over the watershed.}\end{aligned}$$

Horton's equation in the form of Equation (2-42) suffers from the fact that infiltration capacity decreases as a function of time regardless of the actual amount of water available for infiltration. That is, the equation assumes ponding on the surface and a reduction of infiltration capacity, regardless of whether or not the rainfall intensity i exceeds the computed value of infiltration capacity f_0 . For example, it is common that the infiltration capacity of sandy soils greatly exceeds most rainfall intensities, with values of f_0 up to 23 in./hr (Table 2-3). Even intense rainfall pulses are seldom this high, with the consequence that all rainfall may infiltrate; that is, $f = i$. The infiltration capacity should be reduced in proportion to the cumulative infiltration volume, not in proportion to time.

The cumulative infiltration volume is given by the integral of Equation (2-42),

$$F(t) = f_c t + \left[\frac{f_0 - f_c}{k} \right] (1 - e^{-kt}). \quad (2-43)$$

To find the infiltration capacity at any time, Equation (2-42) may be solved iteratively for a time t_p as a function of F . The time t_p is used to establish the appropriate infiltration capacity for the next time interval. This procedure is used, for example, in the SWMM (Huber and Dickinson, 1988), and details can be found there and in Viessman and Lewis (1996).

Table 2-3 Typical Values of the Parameters of f_0 , f_c , and k of the Horton Model

Soil Type	f_c (in./hr)	f_0 (in./hr)	k (hr ⁻¹)
Alphalpha loamy sand	1.40	19.00	38.29
Carnegie sandy loam	1.77	14.77	19.64
Dothan loamy sand	2.63	3.47	1.40
Fuquay pebbly loamy sand	2.42	6.24	4.70
Leefield loamy sand	1.73	11.34	7.70
Tooup sand	1.80	23.01	32.71

After Rawls et al., 1983.

Horton parameters for natural soils (e.g., Table 2–3) are generally not applicable to disturbed soils found everywhere in and near constructed (developed) areas. Pitt et al. (1999, 2001) provide some data and guidance on infiltration parameters as a function of commonly measured soil compaction parameters. This also illustrates the importance of local infiltrometer measurements wherever possible.

Other equations have been developed utilizing analytical solutions to the unsaturated flow equation from soil physics (see Section 2.8). Philip (1957), in a classic set of papers, developed two equations of the form

$$f = (0.5)St^{-1/2} + K, \quad (2-44)$$

$$F = St^{1/2} + Kt, \quad (2-45)$$

where

f = infiltration capacity (in./hr),

F = cumulative infiltration volume (in.),

S = sorptivity, a constant related to soil suction potential (in./hr^{1/2}),

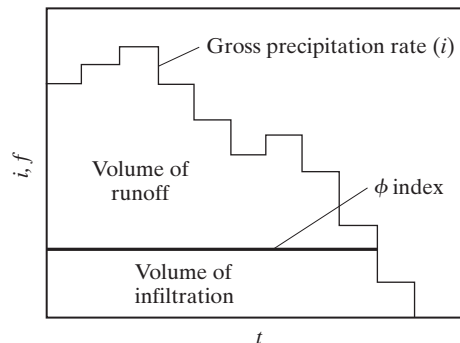
K = soil hydraulic conductivity (in./hr).

Without detailed measurements of actual loss rates and because urban watersheds have high imperviousness, empirical approaches usually give quite satisfactory results. Many urban drainage and flood control studies rely on either the Horton equation or simpler methods to predict average losses during storm passage.

The **ϕ index** is the simplest infiltration method and is calculated by finding the loss difference between gross precipitation and observed surface runoff measured as a hydrograph. The ϕ index method assumes that the loss is uniformly distributed across the rainfall pattern (Fig. 2–17). The shaded area sums to the volume of infiltration, and the area above the ϕ index sums to the volume of direct runoff. Sometimes the method is modified to include a greater initial loss or abstraction followed by a constant loss for the event. The use of ϕ index methods for infiltration is illustrated in Example 2–11.

Figure 2–17

Graphical representation of the phi index method.



PHI INDEX METHOD FOR INFILTRATION**EXAMPLE 2-11**

Use the rainfall data below to determine the ϕ index for a watershed that is 0.875 square miles, where the runoff volume is 228.7 ac-ft.

Time (hr)	Rainfall (in./hr)
0-2	1.4
2-5	2.3
5-7	1.1
7-10	0.7
10-12	0.3

The first step involves graphing the given data, as in Figure E2-10. To approach the problem, we must first change the area of the watershed into acres:

SOLUTION

$$\begin{aligned}\text{area(ac)} &= 0.875 \text{ sq mi (640 acres/sq mi)}, \\ \text{area} &= 560 \text{ acres},\end{aligned}$$

We develop an equation for rainfall minus infiltration for each of the time intervals as given:

$$2(1.4 - \phi) + 3(2.3 - \phi) + 2(1.1 - \phi) + 3(0.7 - \phi) + 2(0.3 - \phi) = 4.9$$

Note that if ϕ is greater than the net rainfall for a specific time period, no negative rainfall is added into the runoff calculation.

The rate of infiltration can be found only by trial and error:

Assume $\phi = 1.5$ in./hr. The runoff is the volume of water above the line at which $y = 1.5$ on the graph in Figure E2-11. This ϕ index would then account for $3(2.3 - 1.5) = 2.4$ in. of runoff (neglecting negative components), which is less than 4.9 in. Try again.

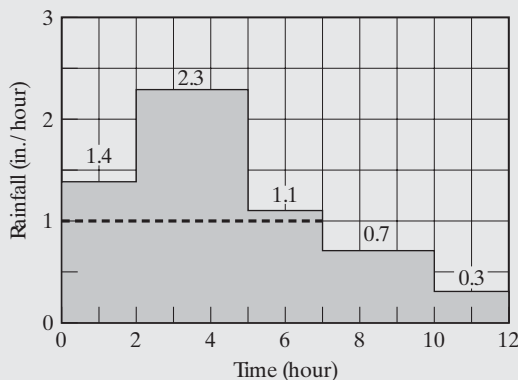


Figure E2-11
Graph of rainfall intensity vs. time.

Assume $\phi = 0.5$ in./hr. This ϕ would account for $2(1.4 - 0.5) + 3(2.3 - 0.5) + 2(1.1 - 0.5) + 3(0.7 - 0.5) = 9.0$ in. of runoff, which is greater than 4.9 in. Try again.

Assume $\phi = 1.0$ in./hr, and the solution is found where the runoff is equal to $2(1.4 - 1.0) + 3(2.3 - 1.0) + 2(1.1 - 1.0) = 4.9$ in., the required amount.

From the calculations, one can see that below the dotted line at which $\phi = 1.0$ in./hr, the rainfall infiltrates into the ground and the rainfall above this line (a total of 4.9 in. in 12 hr) runs off, as required.

More advanced infiltration methods based on actual soil types and solving the governing equations of flow in porous media (i.e., Richard's equation) are presented with examples in the next section. Green and Ampt (1911) assumed a sharp wetting front, separating initial moisture content from saturated moisture content, which has penetrated to a depth L in time t since infiltration began. Water is ponded to a small depth on the surface, and the Green and Ampt model can be used to predict cumulative infiltration as a function of time and soil type. The method requires estimates of hydraulic conductivity, porosity, and wetting front soil suction head. This method has received much attention, since it is based on readily measured parameters for various soil classes.

2.8 GREEN AND AMPT INFILTRATION METHOD

The abstraction term in the water-balance equation usually includes the loss of water through the interception of precipitation on aboveground vegetation, depression storage on the ground surface, and infiltration into underlying soils. As previously discussed, infiltration can be determined using simple empirical methods or theoretically. The infiltration methods that rely on physical rather than empirical parameters provide solutions to the governing equations of flow in porous media (i.e., Darcy's equation presented in Chapter 8) for specific soil types and grain size distributions. This section is a more advanced treatment of infiltration, and it can be skipped without a loss of continuity in the text.

Infiltration is modeled as unsaturated flow and differs from saturated (aquifer) flow because both a liquid (water) and a gas (air) phase are present in the unsaturated system. Gravity and friction are the only forces at work for saturated flow; however, a suction force as discussed in the next paragraph must be included for unsaturated flow. Richard's equation for unsaturated flow in porous media applies the equation of continuity to Darcy's law in order to account for the presence of a gas phase in the subsurface. Richard's equation takes the form:

$$\frac{\partial \theta}{\partial t} = -\frac{\partial}{\partial z} \left[k(\theta) \frac{\partial \psi(\theta)}{\partial z} \right] - \frac{\partial K(\theta)}{\partial z}, \quad (2-46)$$

where

- θ = volumetric moisture content (cm^3/cm^3),
- z = distance below the surface (cm),
- $\psi(\theta)$ = capillary suction (pressure) (cm of water),
- $K(\theta)$ = unsaturated hydraulic conductivity (cm/s).

Moisture content θ is defined as the ratio of the volume of water to the total volume of a unit of porous media. For saturated ground water flow, θ equals the **porosity** of the sample, n , defined as the ratio of interconnected void volume to total sample volume. For unsaturated flow above the water table, $\theta < n$, and remaining void space is filled with air.

The **hydraulic conductivity** K as defined by Darcy's law relates ground water velocity to the hydraulic gradient and is described as the volume of water that will flow through a unit soil column in a given time. Hydraulic conductivity is thus directly proportional to the moisture content of the soil and is usually written as $K(\theta)$ for unsaturated flow.

The **capillary suction** ψ is a measure of the combined adhesive forces that bind the water molecules to solid walls and the cohesive forces that attract water molecules to each other. The capillary suction head draws water to a point at which the tensile suction forces acting on an element of water are in equilibrium with the downward force of gravity acting on the fluid element. The sign of the variable ψ can be thought of in terms of energy or work; ψ is greater than zero in the saturated zone (because positive work is required to push an element of water against hydrostatic pressure) and equals zero at the boundary between the unsaturated and saturated zones. It follows that ψ is less than zero in the unsaturated zone, reflecting the fact that water is held in soil pores under surface-tension forces. During infiltration, the suction acts in the same direction as the water flow and pulls water downward.

Soil physicists refer to $\psi < 0$ as the tension head or **capillary suction**, which can be measured by an instrument called a tensiometer. The capillary term can be added to the elevation head to produce the total hydraulic head acting on an element of water, $h = \psi + z$, where $c = P/\rho g \cdot \psi$ is given as the liquid pressure at a certain point divided by the liquid specific weight; this unit equates to a length term and reiterates that suction head is the relative elevation of a liquid element at a given point with respect to the saturated zone. As opposed to the hydraulic conductivity, the tension head increases with decreasing moisture content and can be written $\psi(\theta)$ for unsaturated flow.

The θ and ψ relationships differ significantly for different types of soil and different saturations; a soil with smaller pores has larger soil surface area acting on an element of water than a large-pored soil. More tension can be applied per unit of water in the smaller pore spaces, allowing water to travel further in the capillary to the point where suction reaches equilibrium with gravity. Greater moisture content permits water molecules to cohere inward to each other rather than adhere to the soil grains, and capillary length decreases. Figure 2-18 summarizes the various unsaturated zone parameters and relationships.

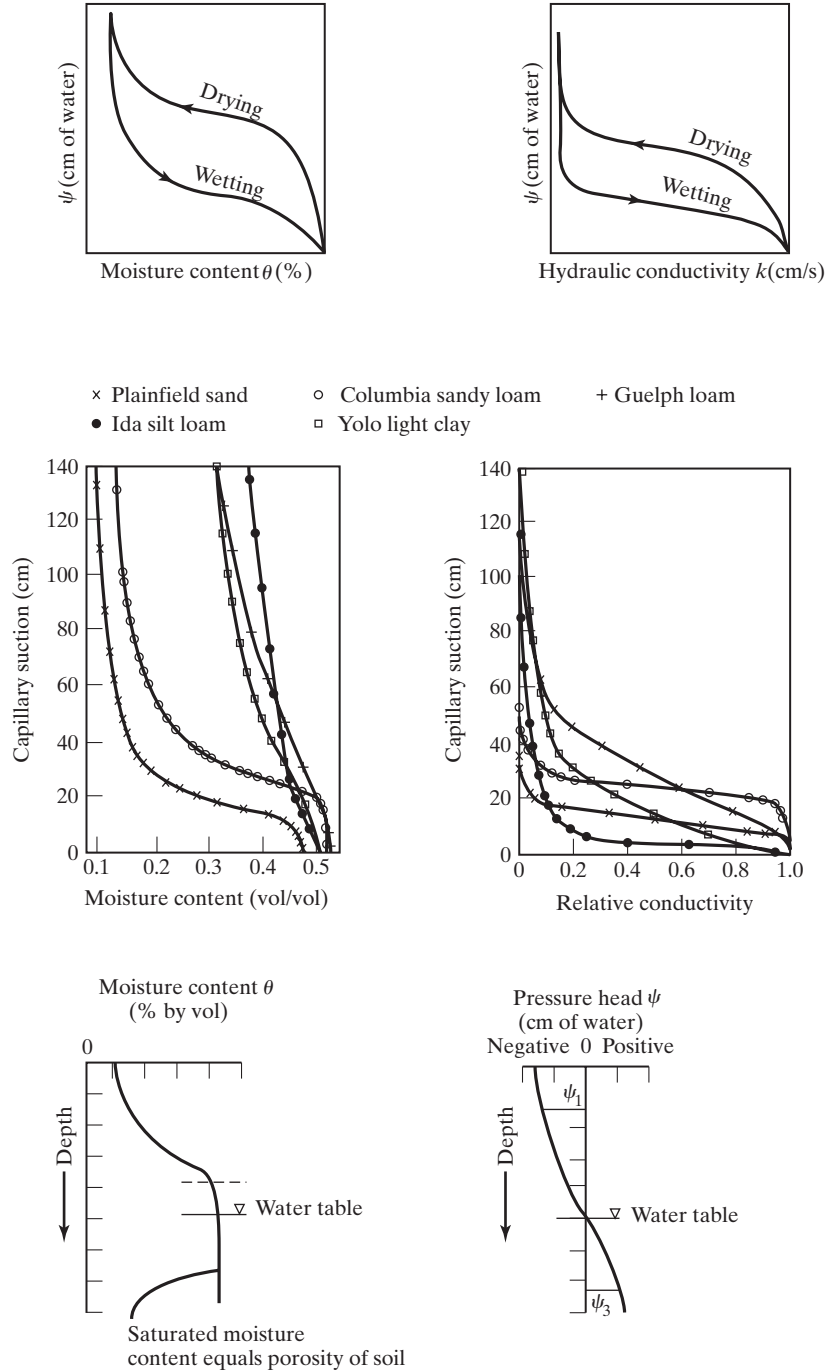
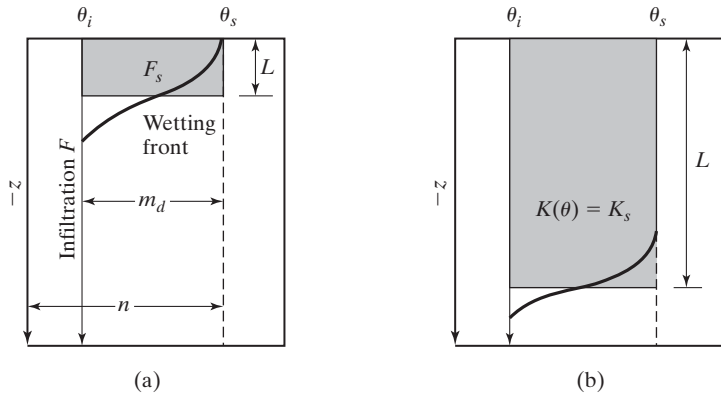
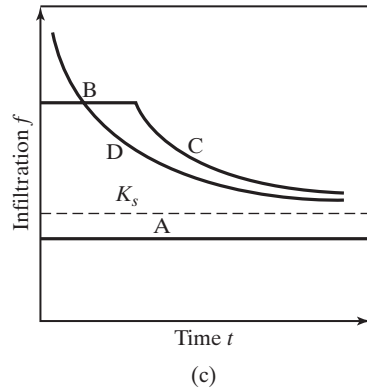


Figure 2-18
Typical relationships in the unsaturated zone.

**Figure 2-19**

Moisture and infiltration relationships. (a) Moisture profile at moment of surface saturation. (b) Moisture profile at a later time. (c) Infiltration behavior under different rainfall. (Adapted from Main and Larson, 1973).



Green and Ampt represents the typical relationship between moisture content and soil depth (Fig. 2-19(a)) as a sharp wetting front separating saturated moisture content at the top of the soil column from the initial moisture content at the column base. The Green and Ampt model is now widely accepted, since it can be used to predict cumulative infiltration as a function of time and readily available soil parameters. For example, it was used as the basis for the analysis of Houston area watersheds after T. S. Allison in 2001 (TSARP, 2001).

The Green and Ampt model operates under five principal assumptions (Ferguson, 1994):

1. The soil under consideration is homogeneous and stable, implying that macropores and preferential migration pathways should not be considered.
2. The supply of ponded water at the surface is not limited (an assumption subject to modification below).

3. A distinct and precisely definable wetting front exists, and as water continues to infiltrate, the wetting front advances at the same rate with depth.
4. The capillary suction just below the wetting front is uniform throughout the profile and constant in time during the infiltration event.
5. The soil is uniformly saturated above the wetting front, and the volumetric water contents remain constant above and below the advancing wetting front.

At the wetting front, the average capillary suction head ψ is used to represent the characteristic moisture curve. The moisture-content profile at the moment of surface saturation is shown in Figure 2-19a. The area above the moisture profile is the amount of infiltration up to surface saturation F and is represented by the shaded area of depth L in Figure 2-19a. Thus, $F = (\theta_s - \theta_i)L = M_d L$, where θ_i is the initial moisture content, θ_s is the saturated moisture content, and $M_d = \theta_s - \theta_i$ is the initial moisture deficit.

The unsaturated hydraulic conductivity of the soil $K(\theta)$ can be substituted into Darcy's law (Chapter 8) to yield:

$$q = -K(\theta) \frac{\partial h}{\partial z}, \quad (2-47)$$

where

$$\begin{aligned} q &= \text{Darcy velocity (cm/s),} \\ z &= \text{depth below surface (cm),} \\ h &= \text{potential or head} = z + \psi \text{ (cm),} \\ \psi &= \text{suction (negative cm),} \\ K(\theta) &= \text{unsaturated hydraulic conductivity (cm/s),} \\ \theta &= \text{volumetric moisture content.} \end{aligned}$$

Equation (2-47) is then applied as an approximation to the saturated conditions between the soil surface (subscript "surf") and the wetting front (subscript "wf"), as indicated in Figure 2-19b:

$$q = -f \cong -K_s(h_{\text{surf}} - h_{\text{wf}})/(z_{\text{surf}} - z_{\text{wf}}), \quad (2-48)$$

in which it is assumed that the Darcy velocity at the soil surface equals the downward infiltration rate $-f$, and the saturated hydraulic conductivity K_s is used to represent conditions between the surface and the wetting front. The depth to the wetting front is L . Thus, with the coordinate z positive upward, $z_{\text{wf}} = -L$. Using the average capillary suction at the wetting front ψ , we have

$$h_{\text{wf}} = z + \psi \cong -L + \psi. \quad (2-49)$$

Noting that $h = 0$ at the surface, Equation (2-48) becomes

$$\begin{aligned} -f &= -K_s[0 - (-L + \psi)]/[0 - (-L)] \\ f &= K_s(1 - \psi/L). \end{aligned} \quad (2-50)$$

The volume of infiltration down to the depth L is given by

$$F = L(\theta_s - \theta_i) = LM_d. \quad (2-51)$$

Substituting for L in Equation (2-50) gives the original form of the **Green–Ampt equation**:

$$f = K_s(1 - M_d\psi/F). \quad (2-52)$$

Equation (2-52) depicts the relationship between the infiltration rate (f) and the cumulative depth of water infiltrated into the soil (F). Remembering that the hydraulic conductivity that transmits water downward based on the hydrostatic pressure is accompanied by the suction head, ψ , which pulls water downward into unsaturated soil, Equation (2-52) indicates that the infiltration rate is a value greater than the saturated hydraulic conductivity as long as there is sufficient water at the surface for infiltration, as sketched in curves B–C and D of Figure 2-19c. The infiltration rate decreases as the cumulative infiltration increases.

Solution of the Green and Ampt equation requires the estimation of three parameters: the vertical saturated hydraulic conductivity (K_s), the moisture deficit (M_d), and the wetting front suction head (ψ). K_s is the most commonly available of the three and is the limiting downward rate of water transmission; for a given fluid, K_s depends on the pore size and grain size properties (Chapter 8). The moisture deficit is a function of the effective porosity and the initial saturation; effective porosity is the space available for water flow (the porosity minus the thin film of water surrounding all soil grains that comprise the residual water content). The suction head is the difference between the atmospheric and hydrostatic pressures.

Three rainfall-infiltration scenarios are possible:

Case 1: $i < K_s$. The rainfall intensity is less than the maximum downward hydraulic conductivity, meaning that runoff will never occur and all rainfall will infiltrate regardless of the duration.

Case 2: $K_s < i < f$. The rainfall intensity is greater than the saturated hydraulic conductivity but less than the infiltration rate. The time to ponding varies for different rainfall intensities.

Case 3: $i > f$. The rainfall intensity is greater than the infiltration rate and runoff can occur.

As mentioned in the discussion of the Horton equation, the rainfall intensity i can be less than the potential infiltration rate given by Equation (2-52), in which case $f = i$. Mein and Larson (1973) showed how the equation can be used to develop the cumulative infiltration F as a function of f . The infiltration is governed by either the rainfall rate or the Green and Ampt function. At the moment of surface saturation, $f = i$, let the corresponding volume of infiltration be F_s . With $f = i$, Equation (2-52) can then be solved for F_s , the volume of infiltration occurring at the time of surface

saturation t_s , when Equation (2-52) becomes valid. Thus, setting $i = f$ and solving for F_s results in the following Green and Ampt equation,

$$F_s = M_d \psi / (1 - i/K_s). \quad (2-53)$$

We require $i > K_s$ in Equation (2-53) and remember that capillary suction ψ is negative. The Green–Ampt infiltration method will predict the following results for various intensities of rainfall i :

1. If $i \leq K_s$, then $f = i$ (curve A in Fig. 2-19(c)),
2. If $i > K_s$, then $f = i$ until $F = F_s = F_s$,
3. Following surface saturation,

$$f = K_s \left(1 - \frac{M_d \psi}{F} \right) \text{ from Equation 2-52 for } i > K_s \text{ and } f = i \text{ for } i \leq K_s.$$

The combined process is sketched in curve B–C of Figure 2-19c. As long as the rainfall intensity is greater than the saturated hydraulic conductivity, the infiltration rate asymptotically approaches K_s as a limiting lower value. Mein and Larson (1973) found excellent agreement between this Green–Ampt method, numerical solutions of Richards' equation, and experimental soils data.

Ponded water descends into the soil due partially to the capillary suction of underlying soil that is drier than the soil near the surface and due to the hydrostatic pressure of the water accumulating on the ground surface. Downward movement stops when all the water that has soaked into the soil and is in equilibrium with the capillary suction of the surrounding soil.

If the rainfall rate starts above, drops below, and then again rises above K_s during the infiltration computations, the use of Green–Ampt becomes more complicated. It is necessary to redistribute the moisture in the soil column, rather than maintaining the assumption of saturation from the surface down to the wetting front shown in Figure 2-19b. The use of the Green–Ampt procedures for unsteady rainfall sequences is illustrated by Skaggs and Khaleel (1982).

Equation (2-52) predicts infiltration rate, f , as a function of cumulative infiltration F , not time. Because $f = dF/dt$, the equation can be converted into a differential equation, the solution of which can then be solved iteratively for $F(t)$.

Application of Green–Ampt Method for Infiltration

A major advantage of the Green–Ampt model is that, in principle, the necessary parameters K_s , ψ , and M_d can be determined from physical measurements in the soil. For example, saturated hydraulic conductivity is tabulated by the U.S. Natural Resources Conservation Service (NRCS, formerly SCS) for a large number of soils as part of that agency's Soil

Properties and Interpretation sheets (available from local NRCS offices and from the web—see Appendix E). An increasing quantity of tension vs. moisture content data (as shown in Fig. 2-18) are also available, from which a value of ψ can be obtained by integration over the moisture content of interest. For example, several volumes of such information have been assembled for Florida soils (e.g., Carlisle et al., 1981). In practice, the Green-Ampt parameters are often calibrated, especially when used in continuous simulation models.

A very useful source of information on Green-Ampt parameters is provided by Rawls et al. (1983), who present data for a large selection of soils from across the United States. These parameters are shown in Table 2-4. Two porosity (θ_s) values are given: total and effective. Effective porosity accounts for trapped air and residual water and is the more reasonable value to use in computations, because it denotes the interconnected pore space available for water flow. It can be seen in Table 2-4 that as the soil particles get finer, from sands to clays, the saturated hydraulic conductivity K_s decreases, the average wetting front suction ψ increases (negatively), and porosity θ_s is variable. Note that K_s also serves as an

Table 2-4 Green-Ampt Infiltration Parameters for Various Soil Texture Classes

Soil Class	Porosity η	Effective Porosity θ_e	Wetting Front Suction Head ψ (-cm)	Hydraulic Conductivity K (cm/hr)
Sand	0.437	0.417	4.95	11.78
	0.374–0.500	0.354–0.480	0.97–25.36	
Loamy sand	0.437	0.401	6.13	2.99
	0.363–0.506	0.329–0.473	1.35–27.94	
Sandy loam	0.453	0.412	11.01	1.09
	0.351–0.555	0.283–0.541	2.67–45.47	
Loam	0.463	0.434	8.89	0.34
	0.375–0.551	0.334–0.534	1.33–59.38	
Silt loam	0.501	0.486	16.68	0.65
	0.420–0.582	0.394–0.578	2.92–95.39	
Sandy clay loam	0.398	0.330	21.85	0.15
	0.332–0.464	0.235–0.425	4.42–108.0	
Clay loam	0.464	0.309	20.88	0.10
	0.409–0.519	0.279–0.501	4.79–91.10	
Silty clay loam	0.471	0.432	27.30	0.10
	0.418–0.524	0.347–0.517	5.67–131.50	
Sandy clay	0.430	0.321	23.90	0.06
	0.370–0.490	0.207–0.435	4.08–140.2	
Silty clay	0.479	0.423	29.22	0.05
	0.425–0.533	0.334–0.512	6.13–139.4	
Clay	0.475	0.385	31.63	0.03
	0.427–0.523	0.269–0.501	6.39–156.5	

Source: Rawls et al., 1983.

estimate for Horton's final infiltration capacity, f_c [Eq. (2-42)]. Table 2-4 provides valuable estimates for Green-Ampt parameters, but local data (e.g., Carlisle et al., 1981) are preferable if available. Missing is the initial moisture content, since it depends on antecedent rainfall and moisture conditions. Ideally, $M_d = n - \theta_i$ should be measured, but this is rarely accomplished. A lower bound for maximum M_d may be inferred from the "available water capacity, in./in." found on NRCS Soil Properties and Interpretation sheets. Available water capacity on these sheets is the difference between field capacity and wilting point, with typical values usually range from 0.05 to 0.30 in./in. These are lower bounds on the maximum M_d , since they do not include water stored above field capacity. The value of M_d to use for a particular soil in question is best determined from a soil test. Otherwise, a conservative (low) M_d value could be used for design purposes (e.g., 0.10). M_d values must be updated in continuous simulation hydrologic models that account for soil moisture content.

In areas of high water tables, there is a limit to the soil storage capacity, and infiltration cannot continue indefinitely without complete saturation of the soil. In such cases, infiltration ceases, losses (rainfall abstractions) become zero, and rainfall excess intensity equals rainfall intensity. If site-specific information is available, this capacity, S , can be estimated from soil moisture data and depth to water table, L , as implied in Figure 2-19b,

$$S = L(\theta_s - \theta_i), \quad (2-54)$$

where L is now the depth to the water table. In some localities, regional information on available soil storage has been prepared (e.g., South Florida Water Management District, 1987). The above value of S is essentially the same as the storage value in the SCS methods described earlier.

The examples below demonstrate the use of Green and Ampt equations with actual soils data. An Excel spreadsheet is available on the textbook website for use in solving for infiltration f as a function of time.

EXAMPLE 2-12

GREEN AND AMPT INFILTRATION EQUATION

For the following soil properties, develop a plot of infiltration rate f vs. infiltration volume F using the Green and Ampt equation:

$$K_s = 1.97 \text{ in./hr},$$

$$\theta_s = 0.518,$$

$$\theta_i = 0.318,$$

$$\psi = -9.37 \text{ in.},$$

$$i = 7.88 \text{ in./hr}.$$

Noting that $M_d = \theta_s - \theta_i$, we can solve Equation (2-53) to obtain the volume of water that will infiltrate before surface saturation is reached:

SOLUTION

$$\begin{aligned}
 F_s &= \frac{\psi M_d}{(1 - i/K_s)} \\
 &= \frac{-(9.37 \text{ in.})(0.518 - 0.318)}{1 - [(7.88 \text{ in./hr})/(1.97 \text{ in./hr})]} \\
 F_s &= 0.625 \text{ in.}
 \end{aligned}$$

Until 0.625 in. has infiltrated, the rate of infiltration is equal to the rainfall rate. After that point (surface saturation), the rate of infiltration is given by the equation [Eq. (2-52)]

$$f = K_s(1 - M_d\psi/F).$$

Solving this equation for various values of F gives the graph shown in Figure E2-12, where f decreases as F increases.

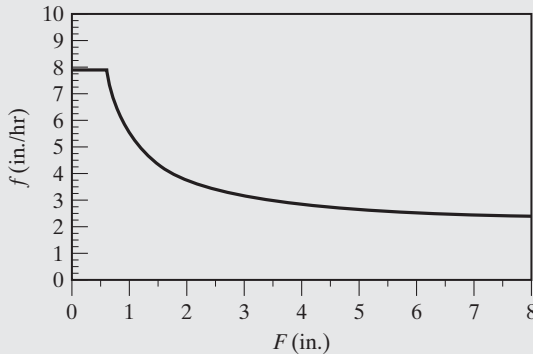


Figure E2-12
Infiltration rate vs.
infiltration volume.

GREEN-AMPT TIME TO SURFACE SATURATION**EXAMPLE 2-13**

Silt loam has the following soil properties for use in the Green-Ampt equation:

$$K_s = 1.81 \times 10^{-4} \text{ cm/sec,}$$

$$\theta_s = 0.523,$$

$$\psi = -17.0 \text{ cm water.}$$

For an initial moisture content of $\theta_i = 0.3$, what is the ponding depth (F_s) and ponding time (t_s) based on this rainfall intensity? What will be the infiltration depth 20 minutes after the beginning of the rainfall? (Assume that the rainfall has a constant intensity.)

$$i = 6K_s \text{ for 10 minutes.}$$

SOLUTION

The initial moisture deficit $M_d = 0.523 - 0.300 = 0.223$. In order to obtain the ponding time, we compute the ponding depth required to produce saturation from Equation (2-53):

$$F_s = (-17.0 \text{ cm})(0.223)/(1 - 6K_s/K_s) = 0.76 \text{ cm}.$$

The ponding time (t_s) is

$$t_s = F_s/i = (0.76 \text{ cm})/[(6 \times 1.81 \times 10^{-4} \text{ cm/s})(60 \text{ s/min})] = 11.7 \text{ min}$$

It shows that it takes rainfall 11.7 minutes to saturate the soil. However, after this point ($11.7 \text{ min} < 20 \text{ min}$), the infiltration depth is not going to be equal to the product of the rainfall intensity (i) and time but increase exponentially according to the equation as follows (Chow et al., 1988):

$$K_s(t - t_p) = F - F_p + \psi M_d \ln \left[\frac{M_d \psi - F}{M_d \psi - F_p} \right]$$

Apparently, infiltration depth (F) varies with time (t) in this equation. There is only one unknown variable (F) in the equation at any time after ponding. We have known $t_s = 11.7 \text{ min}$, $M_d = 0.223$, $\psi = -17.0 \text{ cm}$, $F_s = 0.76 \text{ cm}$, and $t = 20 \text{ min}$. By Goal Seek and rearranging the above equation in Excel spreadsheet to make it equal to zero, the infiltration depth for 20-minute rainfall can be calculated as 1.20 cm. It indicates that 0.44 cm ($1.2 \text{ cm} - 0.76 \text{ cm}$) of rainfall has been infiltrated in 8.3 minutes ($20 \text{ min} - 11.7 \text{ min}$) after the ponding.

2.9 SNOWFALL AND SNOWMELT

A small percentage (about 13%) of precipitation over the entire U.S. falls as snow, but it can be a dominant source of streamflow, especially in mountainous areas during spring melt. This is certainly true of the western United States, but significant percentages of runoff in the northeast and midwest also originate as snow. Because the storage and melting of snow play an important role in the hydrologic cycle of some areas, hydrologists must be able to reliably predict the contribution of snowmelt to overall runoff. Because snowmelt begins in the spring and the derived runoff is out of phase with periods of greatest water demand, control schemes such as storage reservoirs have been implemented in many areas for water supply. Under certain conditions, snowmelt can also contribute to flooding problems, especially in larger river basins with mountainous areas. In addition to forecasting flood situations, snowmelt predictions are also valuable to power companies that generate hydroelectricity and to irrigation districts.

Because of the range of uses for estimating the contribution of snowmelt to streamflow and the variation of conditions applicable to each case, many methods have been developed for computing snowmelt as it affects streamflow. These methods have consistently found that the timing and

amount of runoff depend mainly on three factors: the energy of a snowpack available for melt, the areal extent of the melting snowcover, and the effects of storage on the movement of the meltwater (Male and Gray, 1981).

Snow measurements are obtained using standard and recording rain gages, snow stakes, and snow boards, which measure accumulation over a period of time. Snow surveys at a number of spaced points along a snow course are necessary to account for drifting and blowing snow. The Natural Resources Conservation Service (NRCS) uses devices called snow pillows, which continuously record and transmit the pressure from snow at the bottom of a snowpack. The NRCS has about 500 telemetry sites ("SNOTEL" sites) throughout the western United States transmitting data from snow pillows.

The distribution of mean annual snowfall in the United States is quite variable, and errors may be large in mountainous regions because of the lack of measurements in remote areas. There is a gradual increase in snowfall with latitude and elevation, and from 400 to 500 in./yr (~ 1000 cm/yr) can occur in the Sierra Nevada and Cascade Range of the western United States. Large snows can also occur near the Great Lakes, with average values over 100 in. (254 cm).

Physics of Snowmelt

The energy exchanged between the snowpack, the atmosphere, and the earth's surface is the controlling factor in rates of snowmelt. Other geographic, topographic, and surface cover factors are also important. Density of snow is a major aspect of predicting runoff and is determined by several factors. Snow that is formed under dry, cold conditions will be less dense than snow formed under wet and warmer conditions. As snow accumulates on the ground over a period of time, its density increases with the settling and compaction due to factors such as wind. Wind increases the density of snow in two ways: First, the turbulence breaks ice crystals into smaller ones, making them settle more compactly, and second, drag forces are exerted on the surface of snow due to the moving air and particles. Newly fallen snow has a density about 10% that of liquid water, but the density can increase to 50% with aging. The depth of water that would result if all the snow melted is called the **snow water equivalent, SWE**, and is given by the equation

$$\text{SWE} = 0.01 d_s \rho_s, \quad (2-55)$$

where SWE is in mm, snowdepth d_s is in cm, and ρ_s is in kg/m^3 . The density of snow is often assumed to be 100 kg/m^3 . Hence a rule of thumb for new snow is that the depth of water equivalent is approximately 1/10 the depth of the snowpack. *Ripe snow* holds all the liquid water it can against the action of gravity. If a snowpack is ripe, additional rain can not only result in runoff itself but also warm the snow due to the release of the latent heat when the rain fuses with the pack (Male and Gray, 1981). Hence, any heat added to ripe snow will produce runoff. Heat added to a snowpack at a temperature below freezing will act to warm it.

Snowmelt and evaporation are both thermodynamic processes and can be studied with an energy-balance approach. The energy for snowmelt is derived from the following: (1) net solar radiation, (2) net longwave radiation exchange (from atmosphere and clouds), (3) conduction (diffusion) and convection transfer of sensible heat to or from the overlying air, (4) condensation of water vapor from the overlying air, (5) conduction from the underlying soil, and (6) heat supply by incident rainfall. The heat exchange at the snow-air interface dominates the snowmelt process, and heat exchange with the soil is secondarily important.

For each gram of water melted in a snowpack at 0°C (32°F), 80 cal/g of latent heat must be supplied. Thus, if the density of water is 1 g/cm, 80 cal will melt 80 cm^3 (water equivalent) of snow. Hence, 80 ly (cal/cm²) are required to melt 1 cm depth of snow. Meltwater may refreeze with cooling, or it may drain from the snowpack, contributing to soil moisture, streamflow, or ground water recharge. Snowmelt computations are made difficult by the variations in solar radiation received by the surface and variations in the **albedo**, the reflection coefficient equal to the ratio of reflected to incident shortwave radiation.

Although only 5% to 10% of incoming shortwave radiation is reflected by a water surface, up to 83% is reflected by a clean, dry snow surface. As the snow ages, its albedo can drop to less than 50% because of structural changes, and shortwave radiation penetrates to varying depths, depending on snow density. New precipitation affects the albedo of a snowpack: A snowfall increases the albedo to 0.8 and rain reduces it to 0.4 (Male and Gray, 1981).

Snow radiates essentially as a black body (emissivity = 0.97) according to the Stefan-Boltzmann law, and outgoing longwave radiation at 0°C is equivalent to 8.4 cm (3.3 in.) of melt over a one-day period. Because of re-radiation from the atmosphere back to the earth, longwave radiation loss is equivalent to only about 2 cm (0.8 in.) of melt per day under clear skies and near-freezing temperatures.

Heat exchange between a snowpack and the atmosphere is affected by conduction, convection, condensation, and evaporation (Anderson, 1976; U.S. Army Corps of Engineers, 1956). However, very little evaporation from snow occurs. Direct transformation of snow (ice) to vapor, known as *sublimation*, is an important part of the water balance in the Arctic. Wind velocity and air temperature are primary factors affecting the density of new snow. The rate of transfer of sensible heat (convection) is proportional to the temperature difference between the air T_a and the snow T_s and to the wind velocity v . Condensation melt is proportional to the vapor-pressure difference between the snow surface and the atmosphere and to the wind velocity. The transfer of heat from the underlying soil to the snowpack is small on a daily basis but may accumulate to several centimeters of melt during an entire season. This may be enough to keep the soil saturated and to produce a rapid response of runoff when melting occurs. Raindrop temperature is reduced as it enters a snowpack, and an equivalent amount of heat is imparted to the snow based on the surface wet-bulb temperature. For example, the heat available in 10 mm of rain at 10°C will melt about 1.2 mm of water from the snowpack.

In many basins of very high relief, the lower limit of snowcover (the snow line) is dynamic and moves up and down the slope. As melting proceeds, snowcover recedes more rapidly on southerly and barren slopes, with the rate of snowmelt decreasing with elevation. Given the temperature at an index station, an area–elevation curve for the basin, and the average snow-line elevation, one can compute the area subject to melting based on the rate of change of temperature with elevation, typically about 1°C per 100 m (5°F per 1000 ft) increase in elevation.

The Snowpack Energy Budget

The snowpack energy budget is written in terms of the energy flux to and from the snowpack, with units of energy per unit area per unit time. The most convenient units for Q are cal/cm²-day (ly/day). In terms of the energy budget components mentioned earlier,

$$\Delta H / \Delta t = Q_N + Q_g + Q_c + Q_e + Q_p, \quad (2-56)$$

where

- ΔH = change in heat storage in snowpack,
- Q_N = net (incoming minus reflected minus outgoing) shortwave and longwave radiation,
- Q_g = conduction of heat to snowpack from underlying ground,
- Q_c = convective transport of sensible heat from air to snowpack (negative if from snowpack to air),
- Q_e = release of latent heat of vaporization by condensation of water vapor onto snowpack,
- Q_p = advection of heat to snowpack by rain.

If the change in storage is positive, it will act either to warm the snow, or to ripen the snow, or to produce melt. These are the three phases of snow melt, though they do not always occur in clear, steady sequence (Dingman, 2002). Detailed computation of net shortwave and longwave radiation is discussed in detail in Gray and Prowse, (1993). Convective heat transport and condensation melt are both diffusive processes; the heat flux is proportional to the gradient of temperature for the former and to the gradient of vapor pressure for the latter. Diffusivities or transfer coefficients are functions of wind velocity and surface roughness. Heat transfer to or from the ground Q_g is usually neglected unless the snowpack and ground surface temperatures are both known. Heat advected by rain into the snowpack is simply proportional to the difference between the rain temperature (usually assumed equal to the wet-bulb temperature) and the temperature of the snow.

Using a given time duration and the sum of energy budget components in ly/day ($Q_1 + Q_2 + Q_3 + \dots$), missing variables such as water output can be found from equations for heat storage change (ΔH) (see Dingman, 2002, for examples). The equations for each phase of snowpack development (warming, ripening, and melting) are similar to those of simple melting ice.

First the snow warms from below freezing to 0°C before it can begin melting, according to the equation

$$H_1 = -c_i * \rho_w * \text{SWE} * (T_s - T_m) \quad (2-57)$$

where c_i is the heat capacity of ice (502.1 cal/kg-K), ρ_w is the density of water, SWE is the water equivalent depth of the snow, T_s is the temperature of the snow, and T_m is the melting-point temperature (0°C). This brings the snowpack to the point where it can begin phase change; snow turns to retained water within while maintaining a temperature of 0°C. The pore spaces fill until saturation, and the energy input required is

$$H_2 = \theta_{\text{ret}} * h_s * \rho_w * L \quad (2-58)$$

where θ_{ret} is the maximum volumetric water content that the snow can retain, h_s is the depth of the snow, and L is the latent heat of fusion (79.8 KJ/kg).

Finally, water begins exiting the snowpack as the phase change continues. The heat required for the snow to melt is

$$H_3 = (\text{SWE} - h_{\text{wret}}) * \rho_w * L \quad (2-59)$$

where h_{wret} is the depth of the retained water that is already melted (Dingman, 2002).

Adding H_1 , H_2 , and H_3 results in the change in heat storage necessary to achieve melting. The approximation of 80 ly (cal/cm²) to melt 1 cm depth of snow (from above) takes into account such equations. Assuming the snow is already ripe, the amount of melt from a positive value of ΔH can be found with a shortcut

$$M = \Delta H / 80 \text{ cm/day} \quad (2-60)$$

or

$$M = \Delta H / 203.2 \text{ in./day.}$$

Energy budget methods represent the best techniques for predicting snow-melt when all components can be estimated.

The National Weather Service (Anderson, 1973, 1976) has developed an energy budget model for runoff forecasting. Unfortunately, some energy budget components are usually missing, leading to the use of simpler but less accurate “degree-day methods” based solely on temperature data.

Degree-Day or Temperature-Index Melt Equations

Degree-day or temperature-index equations are empirical or can result from a linearization of the energy budget equation (Huber and Dickinson, 1988). The simplest equation used is

$$M = D_f (T_i - T_B), \quad (2-61)$$

where

M = daily melt as a depth of water equivalent (in./day; mm/day),
 T_i = index air temperature (°F; °C),

T_B = base melt temperature ($^{\circ}\text{F}$; $^{\circ}\text{C}$),

D_f = degree-day melt factor, (in./day- $^{\circ}\text{F}$; mm/day- $^{\circ}\text{C}$).

Empirical degree-day factors range from 0.05 to 0.15 in./day- $^{\circ}\text{F}$ and can be determined by several equations, one of the simplest being

$$D_f = 0.011\rho_s, \quad (2-62)$$

where D_f is in units of mm/day- $^{\circ}\text{C}$ above 0°C and ρ_s is the snow density in kg/m³. This factor is affected by location, time of year, and meteorological conditions, and more complex equations may take into account vegetation transmission for radiation, solar radiation, and snow albedo (Male and Gray, 1981).

Equation (2-61) is basically a regression equation; ACOE (1956) results are shown in Figure 2-20. Either the maximum or average daily temperature for the portion over 0°C is usually used for the index air temperature T_i , and 0°C is most commonly used for T_B (Gray and Prowse, 1993). However, base melt temperatures can be below freezing, since Equation (2-62) represents a linearization about reference conditions well above freezing. Thus, the equations shown in Figure 2-20 should not be used for $T_i < 34^{\circ}\text{F}$. The temperature-index approach is most useful and accurate under normal conditions and less so for extreme conditions. The ACOE developed a number of equations for generalized areas.

The general equation used by the ACOE to calculate the amount of snowmelt accounts for the total amount of energy available for melting the snowpack at a point. The equation is given as:

$$M = Qm334.9\rho_w B \quad (2-63)$$

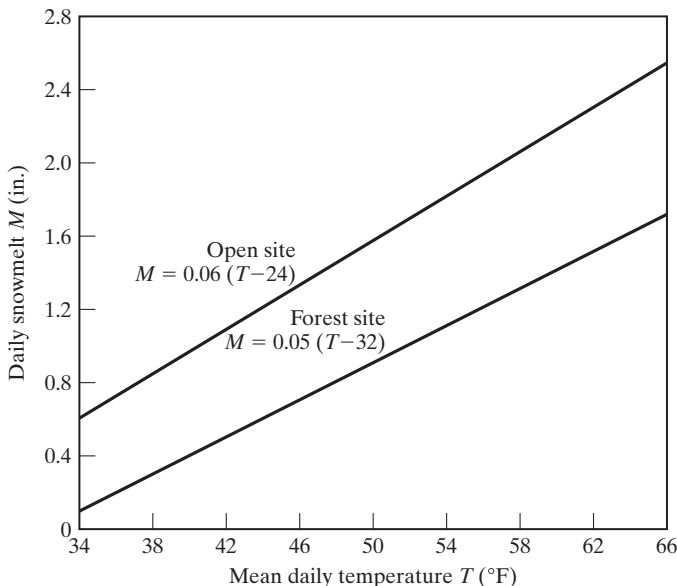


Figure 2-20
Empirical degree-day
equations. (From U.S.
Army Corps of
Engineers, 1956.)

Note: Equations are applicable only to range of temperatures shown on the diagram.

where

- M = snowmelt, mm per day of water equivalent
- Q_m = algebraic sum of all heat components, $\text{kJ/m}^2/\text{day}$
- B = thermal quality of the snow (e.g., ratio of heat required to heat a unit weight of the snow to that of ice at 0°C)
- 334.9 = latent heat of fusion of ice, kJ/kg
- P_w = density of water, kg/m^3

The general equation for snowmelt (2–63) can be combined with long-wave radiation snow melt equations described by ACOE, resulting in a non-linear relationship between temperature and long-wave radiation. These relationships were simplified by fitting linear approximations and shifted to the Fahrenheit temperature scale to derive the following long-wave radiation melt equations.

- a. For melt under clear skies:

$$M_l = 0.0212(T_a - 32) - 0.84 \quad (2-64)$$

- b. For melt under a forest canopy or low cloud cover:

$$M_l = 0.029(T_a - 32) \quad (2-65)$$

where

- M_l = long-wave radiation melt (inches/day)
- T_a = air temperature, F

These are generalized equations used by the ACOE, but other factors such as rain, wind, and percent of forest cover also affect the rate of snowmelt. For further information regarding factors affecting snowmelt and calculating snowmelt rate, please refer to the U.S. Army Corps of Engineers Engineering and Design Manual for Runoff from Snowmelt (1998).

United States Army Corps of Engineers, 1998, Engineering and Design Manual, *Runoff From Snowmelt*, Washington, DC.

Comparison of Model Accuracy

To help compare the energy-balance and temperature-index models, lysimeters are useful for providing actual data in point locations to ensure accuracy, since they collect water from overlying snow and measure outflow over time. In 1976, Anderson set up lysimeters in a clearing in Vermont and observed through six winters the necessary variables of the energy-balance and temperature-index models (Dingman, 2002). The final results show good accuracy for both models, although some years favored one over the other. During the melting phase, Anderson closely matched both snow–water equivalent (SWE) over time and daily water output with lysimeter and visual observation data. Although both models worked well, each requires detailed knowledge of the local environment and changes in conditions. The temperature-index model requires a careful choice of coefficients, and the energy-balance model works



Figure 2-21

Comparison of snow estimation models. (From Anderson, 1976.)

better with the inclusion of more variables. Dingman recommends data on at least air temperature, humidity, wind speed, cloud cover, precipitation, and snow surface. Figure 2.21 shows a typical blizzard in the Northeast, U.S.

Computation of Runoff from Snowmelt

Synthesis of runoff hydrographs associated with snowmelt can be short term, for a few days, or long term, for a complete melt season. Short-term forecasting is used in preparing operational plans for reservoirs or other flood control works and for calculating design floods. To forecast a few days in advance, only the present condition of a snowpack and stream flow are required. Programs used to predict runoff from snowmelt include the ACOE Streamflow Synthesis and Reservoir Regulation (SSARR) and the U.S. NWS River Forecasting System, Snow Accumulation and Ablation Model. For long-term forecasting over a whole season, initial conditions are required as well as reliable predictions of meteorologic parameters.

Snowmelt occurs only from those portions of a watershed covered by snow, and elevation is significant, since rates of snowmelt generally decrease with higher elevation due to a general reduction in temperature. Elevation effects can be considered by dividing a basin into a series of elevation zones, where the snow depth, losses, and snowmelt are assumed uniform in each. In most practical cases, snowmelt is estimated by the index methods presented in the previous section. The budget method itself is used in design flood studies. Once inflow water has been calculated, storage routing techniques are used to accommodate surface and ground water components, each with a different storage time and number of routing units. Storage routing methods are treated in detail in Chapter 4.

SUMMARY

Chapter 2 presents the concept of rainfall–runoff analysis, or the conversion of precipitation to runoff or streamflow, a central problem of engineering hydrology. Gross rainfall must be adjusted for losses to infiltration, evaporation, and depression storage to obtain rainfall excess, which equals direct runoff (DRO).

The UH is defined as 1 in. (1 mm) of direct runoff generated uniformly over a basin for a specified period of rainfall and is a powerful method still used for many watershed studies. The concept of the UH allows for the conversion of rainfall excess into a basin hydrograph, through a linear adding and lagging procedure called hydrograph convolution.

UHs can be derived from actual storm data for gauged basins. Synthetic UHs represent theoretical or empirical approaches that can be applied to ungauged basins. These are most often used for the analysis of watersheds where limited gauging might exist. Most synthetic UH formulas relate parameters such as time to peak or lag time to measures of channel length, slope, watershed size, and watershed shape. A second relation usually correlates peak flow to basin area and lag time. UHs are greatly affected by urban development and channelization, and UH techniques have been customized for local or regional conditions using statistical methods (Chapter 3). Several of the more popular UH methods are presented with examples, including Snyder, SCS, and Clark (TC & R), all contained within the HEC-HMS computer model (Chapter 5).

Kinematic wave methods developed in the 1970s for overland flow hydrographs and channel hydrographs were based on solutions of the continuity and Manning’s equation for a given geometry. Kinematic wave methods are now more useful with the current popularity of numerical methods and advanced hydrologic models, which include HEC-HMS from the U.S. ACOE Hydrologic Engineering Center (2006, 2010) and distributed hydrologic models (Vieux, 2002).

Hydrologic losses are covered in detail in Chapter 2, including evaporation and ET for water bodies and vegetated surfaces. The Penman equation is presented along with a detailed example of its use. Infiltration loss is presented using both simple empirical methods such as Horton and more physics-based methods such as Green and Ampt. Several examples and homework problems are included to cover these methods in detail.

Chapter 2 also presents snowfall and snowmelt theory as it relates to the rainfall–runoff process. The physics of snowmelt is briefly covered, and energy budget and empirical degree-day equations are shown that can be used to estimate melt rates in in./day or cm/day as a function of air temperature, snow temperature, and other factors. Snowmelt can be a significant factor in flood flows and water supply estimates, especially out west in the Rockies and in the Sierra Nevada range in California.

**CONCEPT
CHECK**

- 2.1. What characterizes a watershed?
- 2.2. Describe the four assumptions made that are inherent to the unit hydrograph.
- 2.3. What physical factors affect the shape and timing of the unit hydrograph?
- 2.4. Discuss the parameters that describe the main timing aspects of the hydrograph.
- 2.5. What two characteristics of the hydrograph do most methods for the synthetic unit hydrographs relate?

2.6. In order to determine which of the three excavation sites could have potentially contributed to the sedimentation of the three lakes, it is necessary to determine the watershed boundaries and the contributing drainage areas of the system of lakes. Catchment basins (watersheds) can be determined by connecting ridge lines and dividing lines. Ridge lines will follow the elevation isocontours, and dividing lines are perpendicular to the elevation isocontours at the end and start at the confluences of streams. Using the map on the textbook website displaying the elevation data, determine the following:

- How many confluences are displayed on the map?
- Delineate the watersheds using the elevation isocontours. The total drainage area of the study area is approximately 238 acres.
- Determine which of the three lakes may be affected by accelerated sedimentation.
- Determine which of the three excavation sites may be responsible for the accelerated sedimentation of the lakes.

2.7. A watershed has the following characteristics:

PROBLEMS

$$A = 2600 \text{ ac}$$

$$L = 5 \text{ mi.}$$

$$S = 53 \text{ ft/mi.}$$

$$y = 40\%$$

The channel is lined with concrete.

The watershed is a residential area with 1/4-ac lots. The soil is categorized as soil group B. Assume that the average watershed slope is the same as the channel slope. Determine the UH for this area for a storm duration of 1 hr using the SCS triangular UH method.

2.8. Determine the storm hydrograph resulting from the rainfall pattern in Figure P2-8a using the triangular 1-hr UH given in Figure P2-8b.

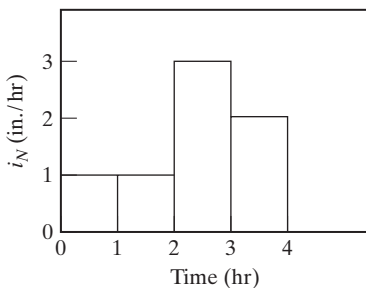


Figure P2-8a
Storm hyetograph.

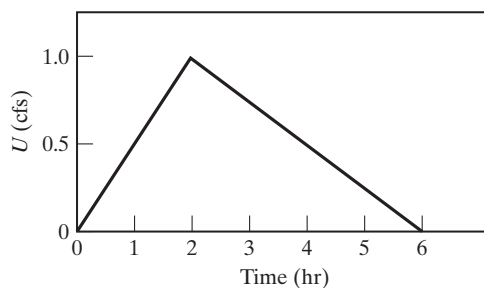


Figure P2-8b
Triangular 1-hr UH.

2.9. (a) Given a triangular 1-hr UH with

$T_B = 12 \text{ hr},$
 $T_R = 4 \text{ hr},$
 $Q_p = 200 \text{ cfs},$

where

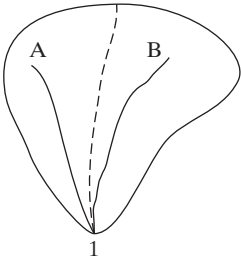
T_B = time base of the UH,
 T_R = time of rise,
 Q_p = peak flow,

develop a storm hydrograph for hourly rainfall (in.) of $P = [0.1, 0.5, 1.2]$.

(b) Repeat the above problem for hourly rainfall (in.) of $P = [0.2, 1.0, 0.4]$.

2.10. A sketch of the Buffalo Creek Watershed is shown in Figure P2–10. Areas A and B are identical in size, shape, slope, and channel length. UHs (1 hr) are provided for natural and fully developed conditions for both areas.

Figure P2–10
Sketch of Buffalo
Creek watershed.



Time (hr)	0	1	2	3	4	5	6	7	8
UH _{dev} (cfs)	0	40	196	300	268	185	90	30	0

Time (hr)	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
UH (nat)	0	12	32	62	108	180	230	182	126	80	53	32	18	6	0

- (a) Assuming natural conditions for both areas, evaluate the peak outflow at point 1 if 2.5 in./hr of rain falls for 2 hr. Assume a total infiltration loss of 1 in.
- (b) Assume that area B has reached full development and area A has remained in natural conditions. Determine the outflow hydrograph at point 1 if a net rainfall of 2 in./hr falls for 1 hr.
- (c) Sketch the outflow hydrograph for the Buffalo Creek Watershed under complete development (A and B both urbanized) for the rainfall given in part (b).

2.11. The 1-hr UH in the accompanying table was recorded for a particular watershed. Determine the size of the watershed in acres and then convert the 1-hr UH into a 3-hr UH for the watershed.

Time (hr)	0	1	2	3	4	5	6	7	8	9	10
U (cfs)	0	6	20	50	80	60	50	20	15	5	0

- 2.12.** A major storm event was recorded for Little Bear Creek. The incremental rainfall and measured hydrograph data for this storm are provided in Table P2-12 in 1-hr increments. The drainage area for the basin is 4.5 mi². Assume that the base flow for Little Bear is zero. (See Fig. 1-26.)

Table P2-12 Little Bear Creek Data

TIME (hr)	Precipitation (in)	Q (cfs)	TIME (hr)	Precipitation (in)	Q (cfs)
15:30	0	0	7:00	0	320
16:00	0	0.4	7:30	0	293
16:30	0.01	0.4	8:00	0	269
17:00	0.05	0.4	8:30	0	246
17:30	0.05	0.5	9:00	0	225
18:00	0.04	0.5	9:30	0	207
18:30	0	0.5	10:00	0	189
19:00	0	0.6	10:30	0	173
19:30	0.37	4.2	11:00	0	159
20:00	0.08	14	11:30	0	146
20:30	0.01	27	12:00	0	133
21:00	0	43	12:30	0	122
21:30	0.05	58	13:00	0	112
22:00	0.42	74	13:30	0	103
22:30	0.18	93	14:00	0	94
23:00	0.1	112	14:30	0	86
23:30	0.28	134	15:00	0	79
0:00	0.3	163	15:30	0	72
0:30	0.15	196	16:00	0	66
1:00	0.06	229	16:30	0	61
1:30	0.14	255	17:00	0	56
2:00	0.19	273	17:30	0	51
2:30	0.23	288	18:00	0	47
3:00	0.50	303	18:30	0	43
3:30	0.11	321	19:00	0	39
4:00	0.01	340	19:30	0	36
4:30	0.01	358	20:00	0	33
5:00	0.01	372			
5:30	0	422			
6:00	0	367			
6:30	0	344			

- (a) Using storm hydrograph, estimate the vol of runoff in inches over the watershed.

- (b) Estimate the vol of infiltration (in.).
- (c) Estimate t_p for the entire event.
- 2.13.** Plot the hydrograph for the storm data given in Problem 1.22 (flow rate vs. time). Label the following:
- (a) peak flow Q_p ,
- (b) time to peak t_p (distance from the center of mass of rainfall to the peak flow),
- (c) time of rise T_R (distance from the start of discharge to the peak flow),
- (d) time base T_B (distance from the start of discharge to the end of discharge).
- 2.14.** Using the convolution equation, develop a storm hydrograph for the rainfall intensity i and infiltration f given in the table (at the end of each time step) using the 30-min unit hydrograph U given below.

Time (hr)	0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
i (cm/hr)	0	1.0	1.25	2.5	1.0						
f (cm/hr)	0	0.75	0.5	0.4	0.3						
U (m ³ /s)	0	33	66	90	75	55	35	20	10	4	0

- 2.15.** Using a program such as Excel, develop the S-curve for the given 30-min UH, and then develop the 15-min UH from the 30-min UH.

Time (hr)	0	0.25	0.5	0.75	1.0	1.25	1.5
U (cfs)	0	15	70.9	118.6	120.0	110	60.9

Time (hr)	1.75	2.0	2.25	2.5	2.75	3.0	3.25
U (cfs)	45.4	33.9	25.3	18.9	14.1	10.5	7.8

Time (hr)	3.5	3.75	4.0	4.25	4.5	4.75	5.0	5.25
U (cfs)	5.8	4.4	3.3	2.4	1.8	1.6	0.8	0

- 2.16.** Using Excel spreadsheet programs, develop the S-curve for the given 3-hr UH, and then develop the 2-hr UH from the 3-hr UH.

Time (hr)	0	1	2	3	4	5
U (cfs)	0	75	180	280	220	220
Time (hr)	6	7	8	9	10	11
U (cfs)	140	60	30	15	5	0

- 2.17.** Given the following 2-hr unit hydrograph, calculate the 1-hr unit hydrograph. Then back calculate and find the 2-hr unit hydrograph to prove that the method of calculation is accurate. Graph both unit hydrographs against time on the same plot.

Time (hr)	0	1	2	3	4	5	6
Flow (cfs)	0	25	120	200	400	500	450

Time (hr)	7	8	9	10	11	12	13
Flow (cfs)	360	300	230	160	100	30	0

- 2.18.** Develop storm hydrographs from UHs of subareas 1 and 2 shown in Figure P2–18 for the given rainfall and infiltration.

t (hr)	i (in./hr)	f (in./hr)
1	0.5	0.4
2	1.1	0.2
3	3	0.2
4	0.9	0.2

Time (hr)	0	1	2	3	4	5	6	7	8	9
UH ₁ (cfs)	0	200	450	650	450	300	150	0		
UH ₂ (cfs)	0	100	300	450	350	250	130	100	50	0

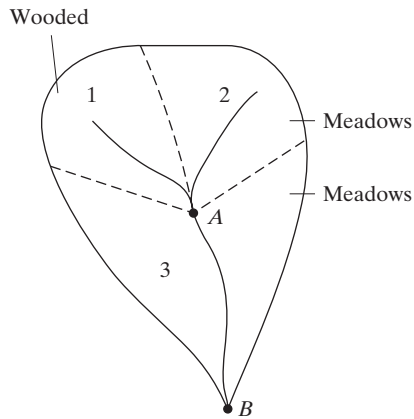


Figure P2–18

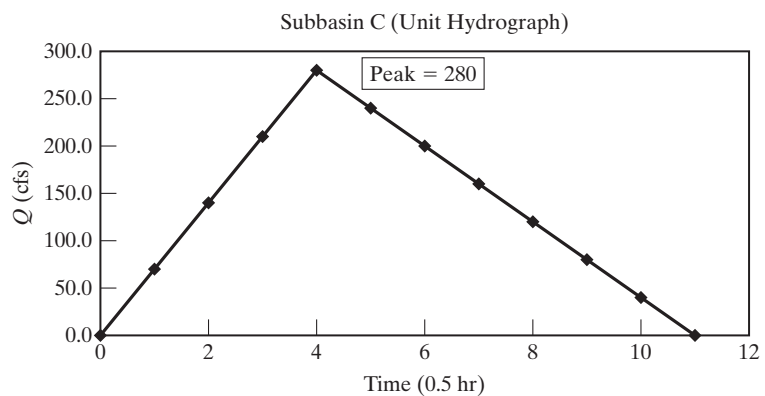
Watershed with three subareas and their respective land use types.

- 2.19.** Develop a combined storm hydrograph at point A in the watershed (Fig. P2–18) and lag route (shift in time only) assuming that travel time from point A to B is exactly 2 hr.
- 2.20.** Develop a storm hydrograph for subarea 3 from the given UH, add to the combined hydrograph from Problem 2.19, and produce a final storm hydrograph at the outlet of the watershed, B.

Time (hr)	UH ₃ (cfs)	t (hr)	i (in./hr)	F (in./hr)
0	0	1	0.5	0.4
1	140	2	1.1	0.2
2	420	3	3	0.2
3	700	4	0.9	0.2
4	490			
5	350			
6	210			
7	130			
8	70			
9	0			

- 2.21.** Redo Example 2–6 if the watershed is soil type B in good cover forest land. How does the forested area compare to the meadow UH?
- 2.22.** Sketch the SCS triangular and curvilinear UHs and the mass curve for a 100-mi² watershed which is 60% good condition meadow and 40% good cover forest land. The watershed consists of 70% soil group C and 30% soil group A. The average slope is 100 ft/mi, the rainfall duration is 3 hr, and the length to divide is 18 mi.
- 2.23.** A small watershed has the characteristics given below. Find the peak discharge Q_p , the basin lag time t_p , and the time base of the unit hydrograph T_b , using Snyder's method.
- $$A = 200 \text{ mi.}^2; C_t = 1.70; L = 27 \text{ mi.}; L_c = 15 \text{ mi.}; C_p = 0.7$$
- 2.24.** For a 55 mi² watershed with $C_t = 2.2$, $L = 15 \text{ mi.}$, $L_c = 7 \text{ mi.}$, and $C_p = 0.5$, find t_p , Q_p , T_b , and D . Plot the resulting Snyder UH.
- 2.25.** Watershed data are provided on Figure 1–22 for a small forested watershed that contains seven subareas as shown. Compute an SCS UH (dimensionless) for subarea B, based on lengths and areas from the watershed. Assume a watershed slope of 0.5% and a $CN = 70$.
- 2.26.** Assume the watershed in Example 2–5 has gone through extensive commercial and industrial growth on the wooded area. Now 50% of the formerly wooded areas have become urbanized, so of that portion, 40% is commercial and business and 60% is fair condition lawn space. Assume the soil is 50% group B and 50% group C for all areas. Using Figure 2–8, determine the runoff volume for a rainfall of 6 in.
- 2.27.** Make sure that the unit hydrograph for subbasin C is a unit hydrograph (subbasin C area = 770 ac). The unit hydrograph for subbasin C is graphed in Figure P2–27.

Figure P2–27
Unit hydrograph for
Subbasin C.



- 2.28.** Using Figure 2–14, find the daily evaporation from a shallow lake with the following characteristics:
- Mean daily temperature = 25.6°C,
Daily solar radiation = 550 cal/cm²,

Mean daily dew point = 4.4°C,

Wind movement (6 in. above pan) = 5.5 ft/s.

- 2.29.** A class A pan is maintained near a small lake to determine daily evaporation (see table). The level in the pan is observed at the end of everyday. Water is added if the level falls *near* 7 in. For each day the difference in height level is calculated between the current and previous day, and the precipitation value is from the current day. Determine the daily lake evaporation if the pan coefficient is 0.70.

Day	Rainfall (in.)	Water Level (in.)
1	0	8.00
2	0.23	7.92
3	0.56	7.87
4	0.05	7.85
5	0.01	7.76
6	0	7.58
7	0.02	7.43
8	0.01	7.32
9	0	7.25
10	0	7.19
11	0	7.08*
12	0.01	7.91
13	0	7.86
14	0.02	7.8

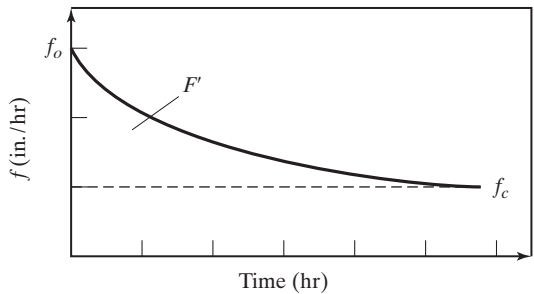
*Refilled at this point to 8 inches

- 2.30.** Given an initial rate of infiltration equal to 1.5 in./hr and a final capacity of 0.5 in./hr, use Horton's equation [Eq. (2-42)] to find the infiltration capacity at the following times: $t = 10$ min, 15 min, 30 min, 1 hr, 2 hr, 4 hr, and 6 hr. You may assume a time constant $k = 0.25/\text{hr}$.
- 2.31.** Determine a Horton equation to fit the following times and infiltration capacities.

Time (hr)	f (in./hr)
1	6.34
2	5.20
6.5	2.50
∞	1.20

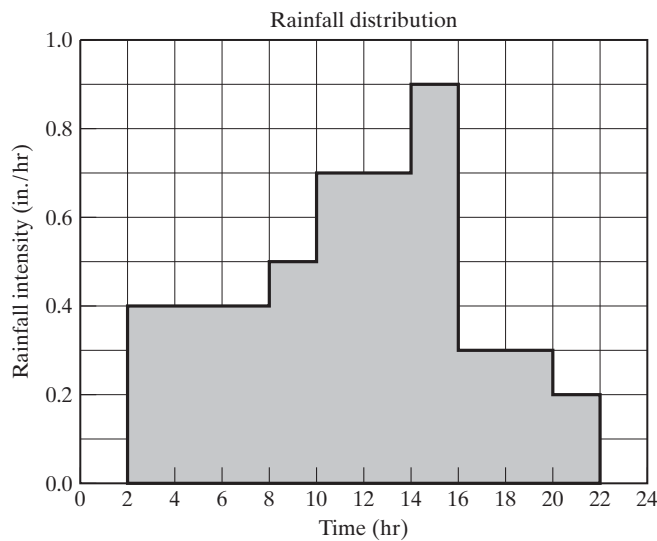
- 2.32.** A 5-hr storm over a 15-ac basin produces a 5-in. rainfall: 1.2 in./hr for the first hour, 2.1 in./hr for the second hour, 0.9 in./hr for the third hour, and 0.4 in./hr for the last 2 hr. Determine the infiltration that would result from the Horton model with $k = 1.1/\text{hr}$, $f_c = 0.2$ in./hr, and $f_o = 0.9$ in./hr. Plot the overland flow for this condition in in./hr vs. t .
- 2.33.** A plot of the infiltration curve obtained using Horton's equation is shown in Figure P2-33. Prove that $k = \frac{(f_o - f_c)}{F'}$ if F' is the area between the curve and the f_c line. Find the area by integration over time, as time approaches infinity.

Figure P2–33
Horton’s infiltration
curve.



2.34. Determine the index of Figure P2–34 if the runoff depth was (a) 5.6 in. of rainfall over the watershed area and (b) 6.5 in.

Figure P2–34
Rainfall distribution.



- 2.35.** A sandy loam has an initial moisture content of 0.18, hydraulic conductivity of 7.8 mm/hr, and average capillary suction of 100 mm. Rain falls at 2.9 cm/hr, and the final moisture content is measured to be 0.45. When does surface saturation occur? Plot the infiltration rate vs. the infiltration volume in 2-35.
- 2.36.** Use the parameters given to graph the infiltration rate vs. the infiltration volume for the same storm for both types of soil. Prepare a graph using the Green–Ampt method, comparing all the curves calculated with both the lower-bound and upper-bound porosity parameters. The rainfall intensity of the storm was 1.5 cm/hr for several hours, and the initial moisture content of all the soils was 0.15.

Soil	Porosity	Capillary Suction (cm)	Hydraulic Conductivity (cm/hr)
Silt loam	0.42–0.58	16.75	0.65
Sandy clay	0.3–0.49	23.95	0.10

- 2.37.** The Green and Ampt infiltration equation is a loss function used to compute the cumulative infiltration, F (cm) for a given infiltration rate, f (cm/hr). Recall that, $f = K_s \cdot (1 - (M \cdot w_f)/F)$. For the given soil properties and infiltration rate, answer the following.

K	1	cm/hr
w_f	-11	cm
M	0.2	

t (hr)	f (cm/hr)	F (cm)
0	0	0
0.01	15.34	0.15
0.25	3.32	0.95
0.50	2.42	1.55
0.75		2.07

- (a) Compute the infiltration rate, f (cm/hr) for $F = 2.07$ cm, and show your computations.
- (b) Compute the cumulative infiltration resulting from a constant rain rate of 0.5 cm/hr for 1 hr.
- (c) At saturation, what is the infiltration rate in cm/hr? Justify.
- 2.38.** Please refer to the Green and Ampt Infiltration and Runoff Example posted on the textbook website along with the associated Excel spreadsheet. Complete the problem in the example, and then repeat the procedure with a constant rainfall rate of 50 mm/hr for 1.8 hrs. Determine the new cumulative runoff and runoff coefficient.

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Chapter 3

Frequency Analysis



Flooding of the Willamette River at Corvallis, Oregon, February 1996. (Photo by Tony Overman. Courtesy Corvallis Gazette-Times, with permission.)



Flooding Occurs Throughout New Orleans as Levees Break Following Hurricane Katrina. (Photo Courtesy Jocelyn Augustino/FEMA.)

3.1 Scope of the Chapter

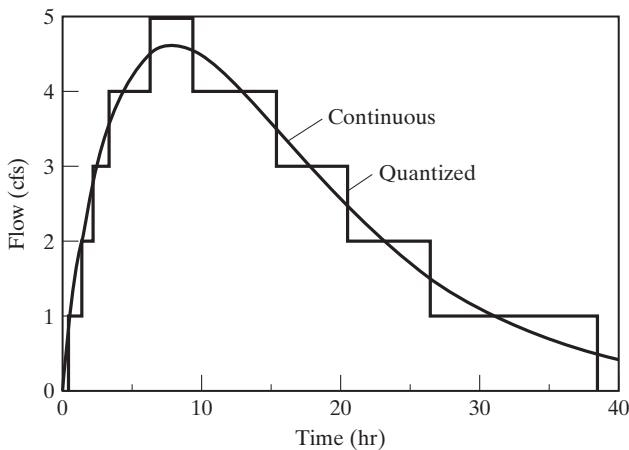
INTRODUCTION

Many processes in hydrology must be analyzed and explained in a probabilistic sense because of their inherent randomness. For instance, it is not possible to predict streamflow and rainfall on a purely deterministic basis in either the past (hindcasting) or the future (forecasting), since it is impossible to know all their causal mechanisms quantitatively. Fortunately, statistical methods are available to organize, present, and reduce observed data to a form that facilitates their interpretation and evaluation. This chapter presents methods by which the uncertainty in hydrologic data may be quantified and presented in a standard probabilistic framework. The primary objective is to relate the magnitude of a hydrologic event, such as a flood, to its probability and vice versa. Within this chapter, the word “flood” is used generically to mean the magnitude of a hydrologic event. Although actual flood data are used in examples, the same techniques may be applied to precipitation and many other hydrologic variables.

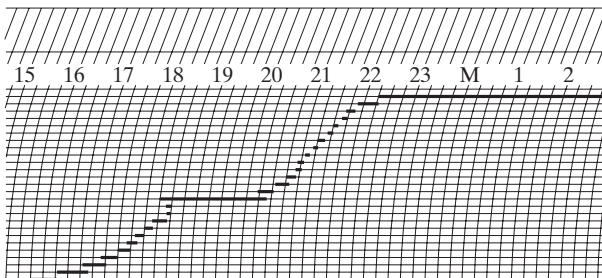
Random Variables

A random variable is a parameter (e.g., streamflow, rainfall, stage) that cannot be predicted with certainty; that is, a random variable is the outcome of a random or uncertain process. Such variables may be treated statistically as either **discrete** or **continuous**. Most hydrologic data are continuous and are analyzed probabilistically using continuous frequency distributions. For example, values of flow in the hydrograph sketched in Figure 3-1(a) can equal any positive real number to the accuracy of the flow meter; that is, the data are continuous. However, the data themselves are often presented in a discrete form because of the measurement process. For example, daily streamflows might be estimated to the nearest cfs. This form of data presentation is called **quantized continuous**; that is, the continuous observations are assigned to discrete ranges. This is also illustrated in Figure 3-1(a), in which flows are assigned to the nearest cfs.

Discrete random variables can take on values only from a specified domain of discrete values. For example, flipping a coin will produce either a head or a tail; rolling a die produces an integer from 1 to 6. The output from a tipping-bucket rain gage is a hydrologic example (Fig. 3-1(b)): There either is



(a)



(b)

Figure 3-1

Continuous and discrete data.
 (a) Continuous and quantized data.
 (b) Discrete data.
 Strip chart output from a tipping-bucket rain gage. Each vertical increment is 0.01 in. of rain.

or is not a tip during any time interval. Quantized-continuous data could be treated as discrete; indeed, they are discretized whenever tabulated data are processed, since numerical values must be truncated (e.g., to the gage accuracy for flow or to the nearest 0.01 in. for rainfall). However, for frequency analysis this is usually very inconvenient because of the enormous range of values that would have to be considered. For example, if streamflows are measured to the nearest cfs and there is a range of flows from 0 to 5000 cfs, then 5000 discrete intervals would have to be considered. Approximating the record as continuous is much easier. Although discrete frequency distributions are sometimes applied to continuous variables (e.g., storm rainfall depths), most applications of discrete distributions in hydrology are to a random variable that represents the *number* of events satisfying a certain criterion—for example, the number of floods expected to exceed a certain magnitude during a period of years.

Presentation of Data

A key assumption of frequency analysis is that the data are **independent**; that is, the magnitude of a future event does not depend upon the magnitude of past events. While underlying climatological mechanisms in hydrology, such as the El Niño–Southern Oscillation (ENSO), may affect long-term flow records, climatological cause-and-effect mechanisms are difficult to quantify (Tootle et al., 2005). A qualitative judgment regarding the presence of a nonrandom pattern in the time series of hydrologic data can often be made on the basis of a plot of the time series (a time series is a record of magnitude vs. the time at which it occurs). For instance, a 75-yr record of the Siletz River near the western Oregon coast is shown in Figure 3–2. A 5-yr running average (the average of the given year and the four preceding years) is also shown, as

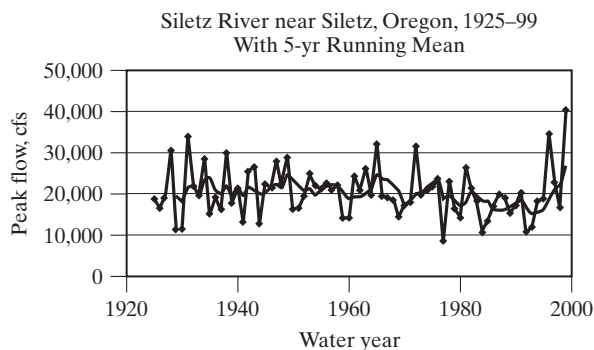


Figure 3–2

Time series of annual maximum peak flows for the Siletz River, near Siletz, Oregon. Also shown is the 5-yr running mean, from which longer-term trends can sometimes be discerned. Only quantitative methods of time-series analysis can determine for sure whether or not there are periodicities or nonstationary components in the data, but none are obvious visually.

a way to dampen out short-term oscillations. There are no obvious visual indicators of flow changes that might indicate changes in the watershed or climatology of the region. Indeed, the Siletz River was chosen for this example largely because its watershed is relatively unaltered. Examples of other possible outcomes leading to **nonstationarity** in the time series are illustrated in Figure 3-3. The statistical descriptors of the data (e.g., mean, variance) change in Figure 3-3(a) due to a linear trend—for example, due to urbanization or other development that increases runoff. Long-term cyclic trends are shown in Figure 3-3(b), perhaps due to climatological changes. The variance of the flows is reduced around 1970 in Figure 3-3(c), perhaps due to construction of a dam upstream of the gage. Finally, the average flow decreases around 1970 in Figure 3-3(d), perhaps due to upstream diversions. Formal methods for evaluating such changes or trends in time series are discussed by Priestly (1981), Helsel and Hirsch (1992), Salas (1993), and Chatfield (2003).

Another possible concern is **mixed populations**—that is, the presence of two or more causative mechanisms for the time series data. For example, floods may be caused by rainfall events, snowmelt events, or a combination of both. The frequency characteristics of events with different causative mechanisms

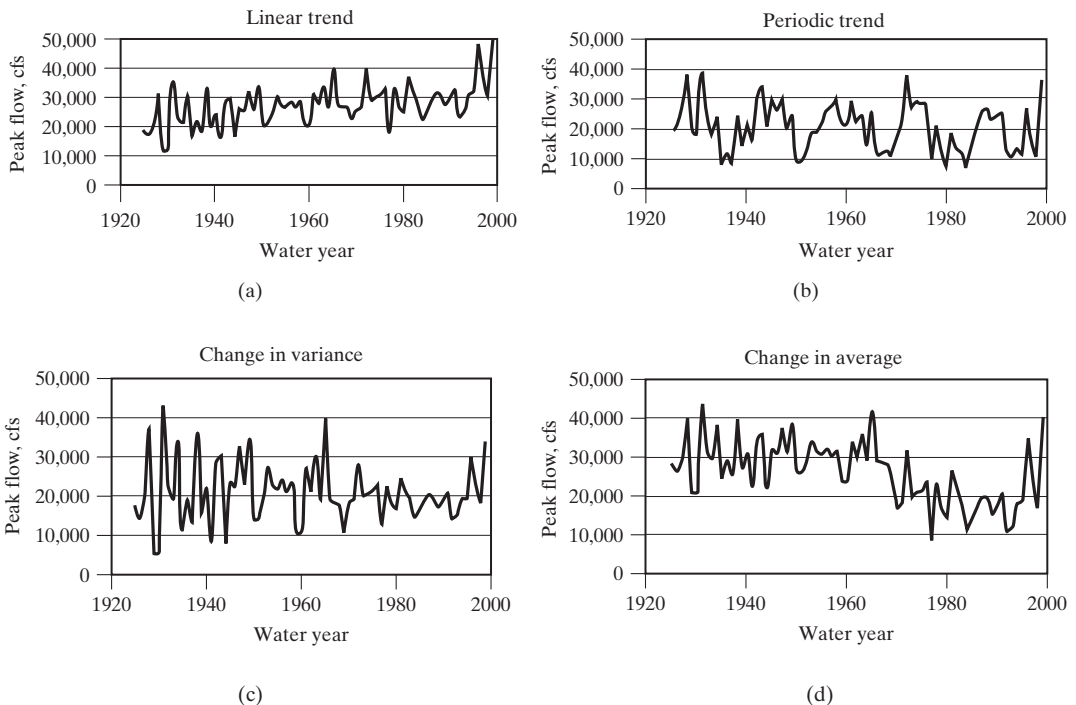


Figure 3-3

Hypothetical examples of nonstationary time series. (a) Linear trend. (b) Periodic trend. (c) Change in variance. (d) Change in average.

can be quite different, although one or the other will usually represent the highest extremes. In such cases, it is common to perform separate frequency analyses on the two types of data, as illustrated by Cudworth (1989).

To aid in the selection of a fitted frequency distribution, quantized-continuous data are often presented in the form of a *bar chart*, or **histogram**. The height and general shape of the histogram are useful for characterizing the data and lend insight into the selection of a frequency distribution to be applied—for example, whether or not the distribution should be symmetric or skewed. Using streamflows as an example, the range of flows is first divided into **class intervals**, and the number of observations (**frequency**) corresponding to each class interval is tabulated. The width of the class intervals should be small enough so that the underlying pattern of the data may be seen but large enough so that such a pattern does not become confusing. The value used for the class interval can alter the viewer's impression of the data (Benjamin and Cornell, 1970); this value may conveniently be altered in many computer plotting programs so that the engineer can compare many different options. As an aid, Panofsky and Brier (1968) suggest

$$k = 5 \log_{10} n, \quad (3-1)$$

where k is the number of class intervals and n is the number of data values. Class intervals do not have to be of constant width if it is more convenient to group some data into a larger, aggregated interval.

If the ordinates of the histogram are divided by the total number of observations, **relative frequencies** (probabilities) of each class interval result, such that the ordinates sum to 1.0, providing an alternative method of plotting the data. Still a third way is in the form of a **cumulative frequency distribution**, which represents a summation of the histogram relative frequencies up to a given interval and is the probability that a value along the abscissa is less than or equal to the magnitude at that point. Both relative frequencies and cumulative frequencies are extensively used in hydrology and are best illustrated by an example.

EXAMPLE 3-1

FREQUENCY HISTOGRAMS

A record of 75 years of annual maximum flows for the Siletz River at Siletz, OR, near the central Oregon coast, is presented in Table 3-1. Equation (3-1) indicates that about nine class intervals would be appropriate. Seven are used here, since this permits convenient boundaries of 5000 cfs. (This criterion of convenient or readable intervals is more important than any rule of thumb for the number of class intervals.) Frequencies and cumulative frequencies are the direct products of Microsoft Excel's Tools/Data Analysis/Histogram function. The "bin" is a table of the upper bounds of each class interval. Relative frequencies and class intervals for plotting (Excel's "category" labels) are added. Relative frequencies and cumulative frequencies are shown in Figures 3-4 and 3-5 using the Excel bar-chart option. For

Table 3-1 Data and Frequency Analysis Computations for the Siletz River, at Siletz, Oregon

Water Year	Peak Flow (cfs)	Water Year	Peak Flow (cfs)	Water Year	Peak Flow (cfs)
1925	18,800	1950	16,400	1975	21,500
1926	16,800	1951	16,600	1976	23,600
1927	19,500	1952	19,400	1977	8630
1928	30,700	1953	24,600	1978	23,100
1929	11,200	1954	21,900	1979	16,600
1930	11,500	1955	21,200	1980	14,500
1931	34,100	1956	22,700	1981	26,500
1932	21,800	1957	20,900	1982	21,400
1933	19,800	1958	22,200	1983	18,300
1934	28,700	1959	14,200	1984	11,300
1935	15,000	1960	14,200	1985	13,600
1936	19,600	1961	24,400	1986	17,100
1937	16,100	1962	20,900	1987	20,000
1938	30,100	1963	26,300	1988	19,200
1939	17,800	1964	19,700	1989	15,400
1940	21,400	1965	32,200	1990	17,200
1941	13,200	1966	19,500	1991	20,500
1942	25,400	1967	19,100	1992	10,800
1943	26,500	1968	18,600	1993	12,000
1944	12,800	1969	14,500	1994	18,300
1945	22,400	1970	17,200	1995	18,800
1946	21,600	1971	18,100	1996	34,700
1947	28,000	1972	31,800	1997	22,700
1948	21,900	1973	19,700	1998	16,800
1949	29,000	1974	20,900	1999	40,500

Data were downloaded for USGS Gage Number 14305500 from the USGS website, <http://water.usgs.gov/nwis/>. The continuous record from 1925 has been used.

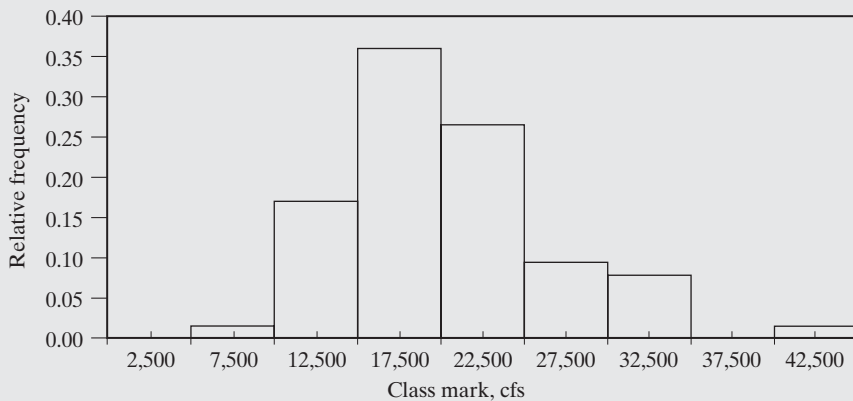
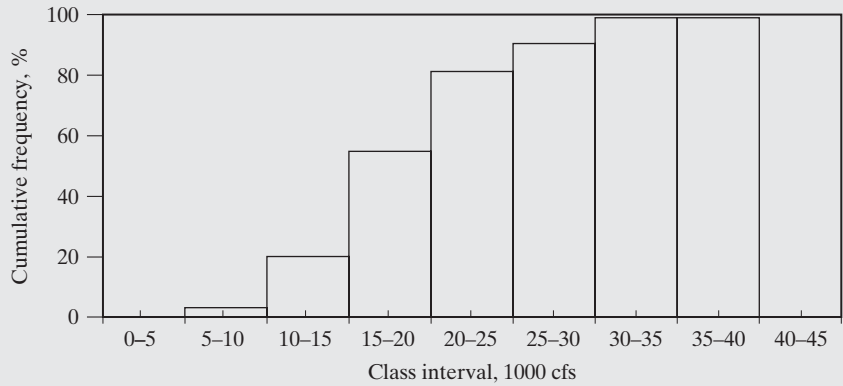
**Figure 3-4** Relative frequencies (probabilities) for the Siletz River, plotted vs. their class mark.

Figure 3–5
Cumulative frequency histogram for the Siletz River, plotted vs. class intervals.



example, in Figure 3–4, relative frequencies are plotted vs. the midpoint of the interval or **class mark**. The probability that a flow lies between 10,000 and 15,000 cfs is 0.17. From the cumulative-frequency histogram (Fig. 3–5), the probability that a flow is less than or equal to 20,000 cfs is 55%. Note that the relative frequencies sum to 1.0 (or 100%), as indicated in Table 3–1 and by the final ordinate shown in Figure 3–5.

Excel Tools/Data Analysis/Histogram Output (First Three Columns)

Bin	Frequency	Cumulative %	Class Interval (cfs)	Class Mark (cfs)	Relative Frequency
5000	0	0.00%	0–5000	2500	0.000
10,000	1	1.33%	5000–10,000	7500	0.013
15,000	13	18.67%	10,000–15,000	12,500	0.173
20,000	27	54.67%	15,000–20,000	17,500	0.360
25,000	20	81.33%	20,000–25,000	22,500	0.267
30,000	7	90.67%	25,000–30,000	27,500	0.093
35,000	6	98.67%	30,000–35,000	32,500	0.080
40,000	0	98.67%	35,000–40,000	37,500	0.000
45,000	1	100.00%	40,000–45,000	42,500	0.013
More	0	100.00%			
Σ =	75			Σ =	1.000

The positive **skew** of the Siletz River flows is apparent (Fig. 3–4); that is, there is a decided “tail” to the right. This will be quantified and related to various distributions in further examples.

3.2 PROBABILITY CONCEPTS

Consider an experiment with N possible outcomes, $X_1, X_2, \dots, X_i, \dots, X_N$. The outcomes are **mutually exclusive** if no two of them can occur simultaneously. They are **collectively exhaustive** if they account for all possible outcomes of the experiment. The **probability** of an event X_i may be defined as

the relative number of occurrences of the event after a very large number of trials. This probability may be estimated as $P(X_i) = n_i/n$, where n_i is the number of occurrences (frequency) of event X_i in n trials. Thus, n_i/n is the relative frequency or probability of occurrence of X_i . (See Fig. 3-4.)

A **discrete probability** is simply the probability of a discrete event. If one defines $P(X_i)$ as the probability of the random event X_i , the following conditions hold on the discrete probabilities of these events when considered over the sample space of N possible outcomes:

$$0 \leq P(X_i) \leq 1, \quad (3-2)$$

$$\sum_{i=1}^N P(X_i) = 1. \quad (3-3)$$

The probability of the **union** (occurrence of either, symbolized by $\cup$) of two mutually exclusive events is the sum of the probabilities of each:

$$P(X_1 \cup X_2) = P(X_1) + P(X_2). \quad (3-4)$$

Two events X_1 and Y_1 are **independent** if the occurrence of one does not influence the occurrence of the other. The probability of the **intersection** (occurrence of both, symbolized by $\cap$) of two independent events is the product of their individual probabilities:

$$P(X_1 \cup Y_1) = P(X_1) \cdot P(Y_1). \quad (3-5)$$

For events that are neither independent nor mutually exclusive,

$$P(X_1 \cup X_2) = P(X_1) + P(Y_1) - P(X_1 \cap Y_1). \quad (3-6)$$

The conditional probability of event X_1 given that event Y_1 has occurred is

$$P(X_1|Y_1) = P(X_1 \cap Y_1)/P(Y_1). \quad (3-7)$$

If events X_1 and Y_1 are independent, then Equations (3-5) and (3-7) combine to give

$$P(X_1|Y_1) = P(X_1) \cdot P(Y_1)/P(Y_1) = P(X_1). \quad (3-8)$$

These concepts are often illustrated on Venn diagrams (Fig. 3-6), on which areas are proportional to probabilities, with the total area corresponding to a probability of 1.0, or 100%.

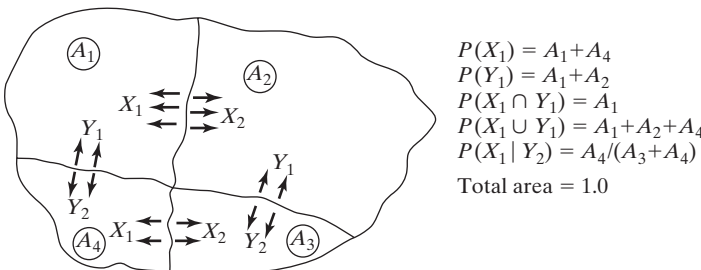


Figure 3-6

Venn diagram for illustration of probabilities.

EXAMPLE 3-2**CONDITIONAL PROBABILITIES**

Let event Y_1 be the condition that a rainstorm occurs on a given day and event X_1 be the condition that lightning is observed on a given day. Let probabilities of these events be

$$P(X_1) = 0.3 \quad (\text{Probability of lightning} = 30\%)$$

$$P(Y_1) = 0.1 \quad (\text{Probability of rain} = 10\%)$$

$$P(Y_1|X_1) = 0.5 \quad (\text{If there is lightning, the conditional probability of rain is } 50\%)$$

What is the probability that *both* rain and lightning occur (i.e., the probability of the intersection of X_1 and Y_1)? From Equation (3-7),

$$P(X_1 \cap Y_1) = P(Y_1|X_1) \cdot P(X_1) = 0.15.$$

But what if both events were independent, such that $P(Y_1|X_1) = P(Y_1) = 0.1$? Then

$$P(X_1 \cap Y_1) = P(X_1) \cdot P(Y_1) = 0.03.$$

The probability of the joint occurrence of independent events will always be less than or equal to the probability of their joint occurrence if they are dependent.

3.3 RANDOM VARIABLES AND PROBABILITY DISTRIBUTIONS

Discrete and Continuous Random Variables

The behavior of a random variable may be described by its probability distribution. Every possible outcome of an experiment is assigned a numerical value according to a discrete **probability mass function (PMF)** or a continuous **probability density function (PDF)**. In hydrology, discrete random variables are most commonly used to describe the *number* of occurrences that satisfy a certain criterion, such as the number of floods that exceed a specified value or the number of storms that occur at a given location. Examples later in this chapter will be of this type. As a rule, discrete random variables are associated with parameters that can assume only integer values. However, it is possible to round continuous variables to the nearest integer or to rescale to integers. For example, 2.18 in. of rain is 218 hundredths of an inch of rain.

Occasionally, it is convenient to treat a continuous variable in a discrete sense, as for the flows described in Figure 3-4. Let the notation $P(x_1)$ mean the probability that the random variable X takes on the value $x_1 \pm \Delta x$. From Figure 3-4, $\Delta x = 2500$ cfs, or half of the class interval. Thus letting X represent “discrete” streamflows, we can assign relative frequencies from Figure 3-4:

$$\begin{aligned} P(7500) &= 0.013, & P(27,500) &= 0.093, \\ P(12,500) &= 0.173, & P(32,500) &= 0.080, \end{aligned}$$

$$\begin{aligned} P(17,500) &= 0.360, & P(37,500) &= 0.000, \\ P(22,500) &= 0.267, & P(42,500) &= 0.013. \end{aligned}$$

Note that these values satisfy the probability axioms of Equations (3-2) and (3-3). Furthermore, for discrete probabilities,

$$P(a \leq x \leq b) = \sum_{a \leq x_i \leq b} P(x_i). \quad (3-9)$$

The **cumulative distribution function (CDF)** is defined as

$$F(x) = P(X \leq x) = \sum_{x_i \leq x} P(x_i). \quad (3-10)$$

From the values tabulated above, $F(22,500) = 0.013 + 0.173 + 0.360 + 0.267 = 0.813$ or 81.3%.

Continuous random variables are usually used to represent hydrologic phenomena, such as flow, rainfall, volume, depth, and time. Values are not restricted to integers, although continuous variables might be commonly rounded to integers. For a continuous random variable, the *area* under the PDF $f(x)$ represents probability. Thus (see Fig. 3-7),

$$P(x_1 \leq x \leq x_2) = \int_{x_1}^{x_2} f(x) dx, \quad (3-11)$$

and the area under the PDF equals 1.0:

$$\int_{-\infty}^{\infty} f(x) dx = 1.0. \quad (3-12)$$

The PDF itself is *not* a probability and has units the inverse of the units of X —for example, cfs^{-1} . However, unlike other engineering calculations, it is not customary to list the units of the PDF. In fact, the numerical value of the PDF is rarely needed. Rather, it is the CDF that is of interest,

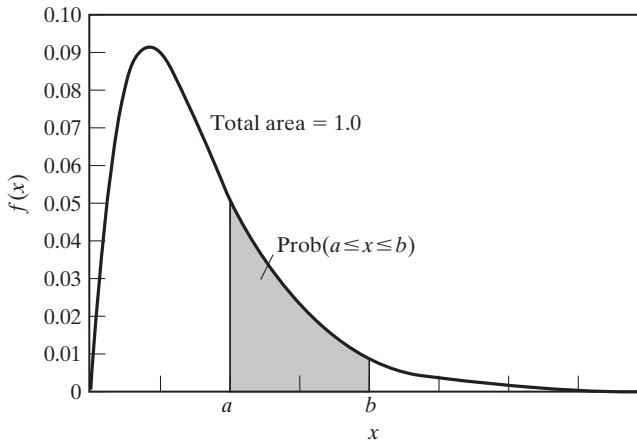


Figure 3-7
Continuous probability density function.

since it is a probability. The continuous CDF is defined similarly to its discrete counterpart:

$$F(x_1) = P(-\infty \leq x \leq x_1) = \int_{-\infty}^{x_1} f(x) dx. \quad (3-13)$$

Other properties include

$$0 \leq F(x) \leq 1.0 \quad (3-14)$$

and

$$P(x_1 < x \leq x_2) = F(x_2) - F(x_1). \quad (3-15)$$

The PDF and CDF are related by Equation (3-13), whose inverse is

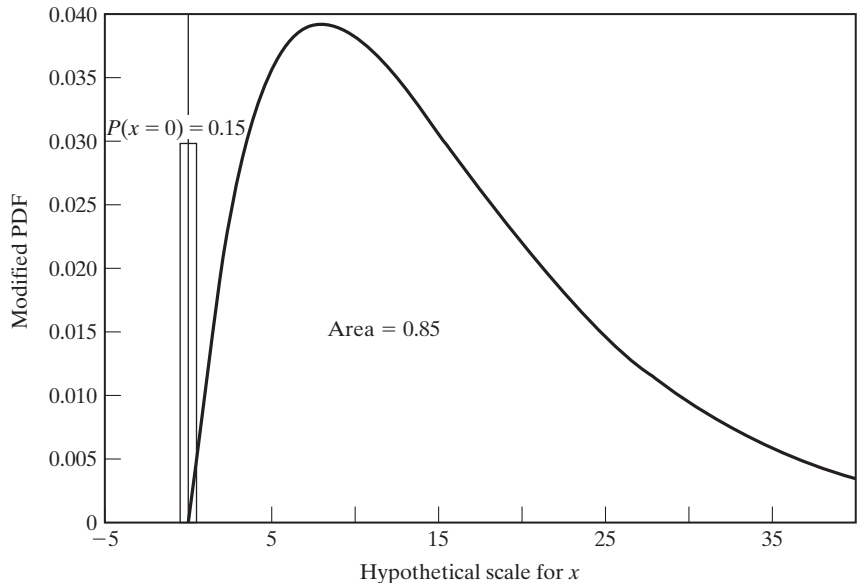
$$\frac{dF(x)}{dx} = f(x). \quad (3-16)$$

The histogram of Figure 3-4 may be represented by a continuous PDF if the relative frequencies are divided by the class interval Δx . The area under the histogram is then 1.0. For example, if the ordinates of the relative-frequency histogram of Figure 3-4 were divided by 5000 cfs, the figure would correspond to a PDF. This illustrates that PDFs may have any nonnegative and single-valued shape; they need not look like smooth curves.

It is possible to have mixed distributions in which a discrete probability mass represents the probability that a variable takes on a specific (discrete) value, while a continuous PDF represents the rest of the range, with an area equal to 1.0 minus the discrete probability mass. For example, a mixed distribution is shown in Figure 3-8, in which the probability is 0.15 that the flow is zero.

Figure 3-8

Mixed frequency distribution. Use discrete PMF for probability of zero value and continuous PDF (scaled to area = 0.85) for probabilities for values greater than zero.



The choice of which continuous PDF to use to represent the data is difficult, since several candidates often are able to mimic the shape of the frequency histogram (Fig. 3-4). Representative PDFs commonly used for hydrologic variables will be described subsequently.

Moments of a Distribution

The concept of moments is common from engineering mechanics. A PMF or PDF is a functional form whose moments are related to its parameters. Thus, if moments can be found, then generally so can the parameters of the distribution. The moments themselves are also indicative of the shape of the distribution.

For a discrete distribution, the N th moment about the origin can be defined as

$$\mu'_N = \sum_{i=-\infty}^{\infty} x_i^N P(x_i) \quad (3-17)$$

and for a continuous distribution as

$$\mu'_N = \int_{-\infty}^{\infty} x^N f(x) dx. \quad (3-18)$$

The first moment about the origin is the mean or average or expected value, denoted by $E(x)$ for “expectation.” Thus,

$$E(x) \equiv \mu = \sum_{i=-\infty}^{\infty} x_i P(x_i) \quad (\text{for a discrete PMF}) \quad (3-19)$$

and

$$E(x) \equiv \mu = \int_{-\infty}^{\infty} x f(x) dx \quad (\text{for a continuous PDF}). \quad (3-20)$$

The mean is a measure of central tendency and is also called a *location parameter*, because it indicates where along the x -axis the bulk of the distribution is located.

Often the distribution of one variable will be known, and information will be desired about a related variable. For example, the distribution of flows may be known and information desired about stage, which is a function of flow. The expected value of a function $g(x)$ of a random variable x may be defined in a similar manner as for the original variable x :

$$E[g(x)] = \sum_{i=-\infty}^{\infty} g(x_i) P(x_i) \quad (3-21)$$

when x is a discrete random variable, and

$$E[g(x)] = \int_{-\infty}^{\infty} g(x) f(x) dx \quad (3-22)$$

when x is a continuous random variable. The expectation is a linear operator, such that if a and b are constants,

$$E(a) = a, \quad (3-23)$$

$$E(bx) = bE(x), \quad (3-24)$$

and

$$E(a + bx) = a + bE(x). \quad (3-25)$$

Higher-order moments about the origin of distributions are usually not needed. Instead, **central moments** about the mean may be defined for a discrete PMF as

$$\mu_N = \sum_{-\infty}^{\infty} (x_i - \mu)^N P(x_i) \quad (3-26)$$

and for a continuous PDF as

$$\mu_N = \int_{-\infty}^{\infty} (x - \mu)^N f(x) dx. \quad (3-27)$$

These moments are simply the expected value of the difference between x and the mean, raised to the N th power. Clearly, the first central moment is zero. The second central moment is called the **variance** and is very important:

$$\text{Var}(x) \equiv \sigma^2 = \mu_2 = E(x - \mu)^2 = \sum_{-\infty}^{\infty} (x_i - \mu)^2 P(x_i) \quad (3-28)$$

for discrete random variables, and

$$\text{Var}(x) \equiv \sigma^2 = \mu_2 = E[(x - \mu)^2] = \int_{-\infty}^{\infty} (x - \mu)^2 f(x) dx \quad (3-29)$$

for continuous random variables. The variance is the expected value of the squared deviations about the mean and represents the *scale*, or spread, of the distribution. An equivalent measure is the **standard deviation**, σ , which is simply the square root of the variance. From the properties of the expectation,

$$\begin{aligned} \text{Var}(x) &= E[(x - \mu)^2] = E(x^2 - 2x\mu + \mu^2) \\ &= E(x^2) - E(2\mu x) + E(\mu^2) \\ &= E(x^2) - 2\mu E(x) + \mu^2 = E(x^2) - 2\mu^2 + \mu^2 \\ &= E(x^2) - \mu^2 \\ &= E(x^2) - [E(x)]^2. \end{aligned} \quad (3-30)$$

The variance is *not* a linear operator. Useful relationships include

$$\text{Var}(a) = 0, \quad (3-31)$$

$$\text{Var}(bx) = b^2 \text{Var}(x), \quad (3-32)$$

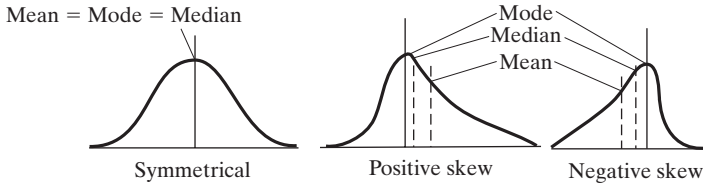


Figure 3-9
Effect of skewness
on PDF and relative
locations of mean,
median, and mode.

$$\text{Var}(a + bx) = b^2 \text{Var}(x), \quad (3-33)$$

where a and b are constants.

Higher moments can be defined if needed, but the only one commonly used in hydrology is the **skewness**, which is the third central moment normalized by dividing by the cube of the standard deviation:

$$g \equiv \mu_3 / \sigma^3. \quad (3-34)$$

The skewness is a *shape* parameter and is illustrated in Figure 3-9. If the distribution is symmetric, the skewness is *zero*.

It is sometimes useful to have a normalized measure of the scale of the distribution. The coefficient of variation, defined as the ratio of the standard deviation to the mean, may be used for this purpose:

$$CV = \sigma / \mu. \quad (3-35)$$

An additional measure of central tendency is the **median**, x_m , which is not a moment but rather the value of x for which the CDF equals 0.5:

$$F(x_m) = 0.5. \quad (3-36)$$

Although not a moment, another parameter of occasional interest is the **mode** of the distribution. This is the value of x at which the PDF (or PMF) is a maximum. The relationship between the mean, median, and mode is also illustrated in Figure 3-8. Most distributions of interest are unimodal. (The mixed distribution of Fig. 3-8 is bimodal.)

The moments and parameters discussed in this subsection refer to the underlying probability distributions and may be derived analytically. Functional forms for PMFs or PDFs can be substituted into the summations or integrals and the moments evaluated in terms of the parameters of the distribution (Benjamin and Cornell, 1970). This task is not illustrated here, since the relationships between moments and distribution parameters will be given for each distribution discussed. The relationships provide a simple method of obtaining estimates of distribution parameters if the moments are known. For this purpose, estimates of the moments must be made from the data.

Estimates of Moments from the Data

Given numerical values for the parameters of a distribution, it is possible to generate a series $x_1, x_2, \dots, x_n$ of random variables that belong to a

given PMF or PDF. Such a series of infinite length would constitute the **population** of all random variables belonging to a given PMF or PDF with a given set of parameters. Similarly, the parameters define the moments, since they are related, as discussed above (and demonstrated subsequently). Observed hydrologic data often result from a mixture of physical processes (e.g., runoff may result from a mixture of rainfall and snowmelt) and hence may incorporate a mixture of underlying probability distributions. In addition, observed data are ordinarily subject to errors of observation and will not fit any distribution perfectly. Hence, the underlying population values of the moments calculated from such data will remain unknown. However, *estimates* of their values may readily be obtained from the data, as shown below for the three moments of most importance in hydrology. If the number of independent samples of a random variable is n , an estimate of the mean (function AVERAGE in Excel) is

$$\hat{\mu} = \bar{x} = \frac{1}{n} \sum_{i=1}^n x_i. \quad (3-37)$$

Higher moments are subject to **bias** in their estimates. An unbiased estimate is one for which the expected value of the estimate equals the population value. It can be shown (Benjamin and Cornell, 1970) that $E(\bar{x}) = \mu$, as desired. For the variance, an unbiased estimate (the “sample estimate”) is

$$\begin{aligned} \hat{\sigma}^2 \equiv S^2 &= \frac{1}{n-1} \sum (x_i - \bar{x})^2 \\ &= \frac{\sum x_i^2 - n\bar{x}^2}{n-1}, \end{aligned} \quad (3-38)$$

where the divisor $n-1$ (instead of the intuitive value n) eliminates the bias. This is function VAR in Excel. (Unless stated otherwise, all summations are from 1 to n .) Clearly, for large samples, a divisor of n (the “population estimate”) would yield almost the same answer, and both divisors may be found in practice, although the unbiased estimate is usually preferred. Computationally, the second form of Equation (3-38) is often preferable.

Since the moment estimates are functions of random variables, they themselves are random variables. The variance of the estimate of the mean may be computed to be

$$\text{Var}(\bar{x}) \equiv S_{\bar{x}}^2 = \frac{S_x^2}{n}. \quad (3-39)$$

Thus, if the variance of the mean is interpreted as a measure of the error in the estimate of the mean, it is reduced as the sample size increases. This is generally true for all moment estimates.

The skewness presents special problems, since it involves a summation of the cubes of deviations from the mean and is therefore subject to larger errors in its computation. An approximately unbiased estimate is

$$C_s \equiv \hat{g} = \frac{n}{(n-1)(n-2)} \cdot \frac{\sum (x_i - \bar{x})^3}{S_x^3} \quad (3-40)$$

with S_x^3 given by Equation (3-38). (This is function SKEW in Excel.) Unfortunately, the appropriate bias correction depends on the underlying distribution; various corrections derived from hydrologic data are available (Bobee and Robitaille, 1975; Tasker and Stedinger, 1986). The skewness estimate computed using Equation (3-40) is called the **station estimate**, meaning that the estimate incorporates data values only from the gaging station of interest.

Error and bias in the skewness estimate increase as the number of observations n decreases. The “Bulletin 17B method” (Interagency Advisory Committee on Water Data, or IACWD, 1982) uses a generalized estimate of the coefficient of skewness, C_w , based on the equation

$$C_w = WC_s + (1 - W)C_m, \quad (3-41)$$

where W is a weighting factor, C_s is the coefficient of skewness computed using the sample data [Eq. (3-40)], and C_m is a regional skewness, which is determined from a map such as in Figure 3-10. The weighting factor W is calculated to minimize the variance of C_w , where

$$W = \frac{V(C_m)}{V(C_s) + V(C_m)}. \quad (3-42)$$

Determination of W requires knowledge of variance of $C_m[V(C_m)]$ and variance of $C_s[V(C_s)]$. $V(C_m)$ has been estimated from the map of skew coefficients for the United States as 0.302 (IACWD, 1982). By substituting W into Equation (3-41), the weighted skew C_w can be written

$$C_w = \frac{V(C_m)C_s + V(C_s)C_m}{V(C_m) + V(C_s)}.$$

The variance of the station skew C_s for log Pearson type 3 random variables can be obtained from the results of Monte Carlo experiments by Wallis et al. (1974). They showed that

$$V(C_s) = 10^{A-B \log_{10}(n/10)},$$

where

$A = -0.33 + 0.08 C_s $	if	$ C_s \leq 0.90$ or
$A = -0.52 + 0.30 C_s $	if	$ C_s > 0.90,$
$B = 0.94 - 0.26 C_s $	if	$ C_s \leq 1.50$ or
$B = 0.55$	if	$ C_s > 1.50,$

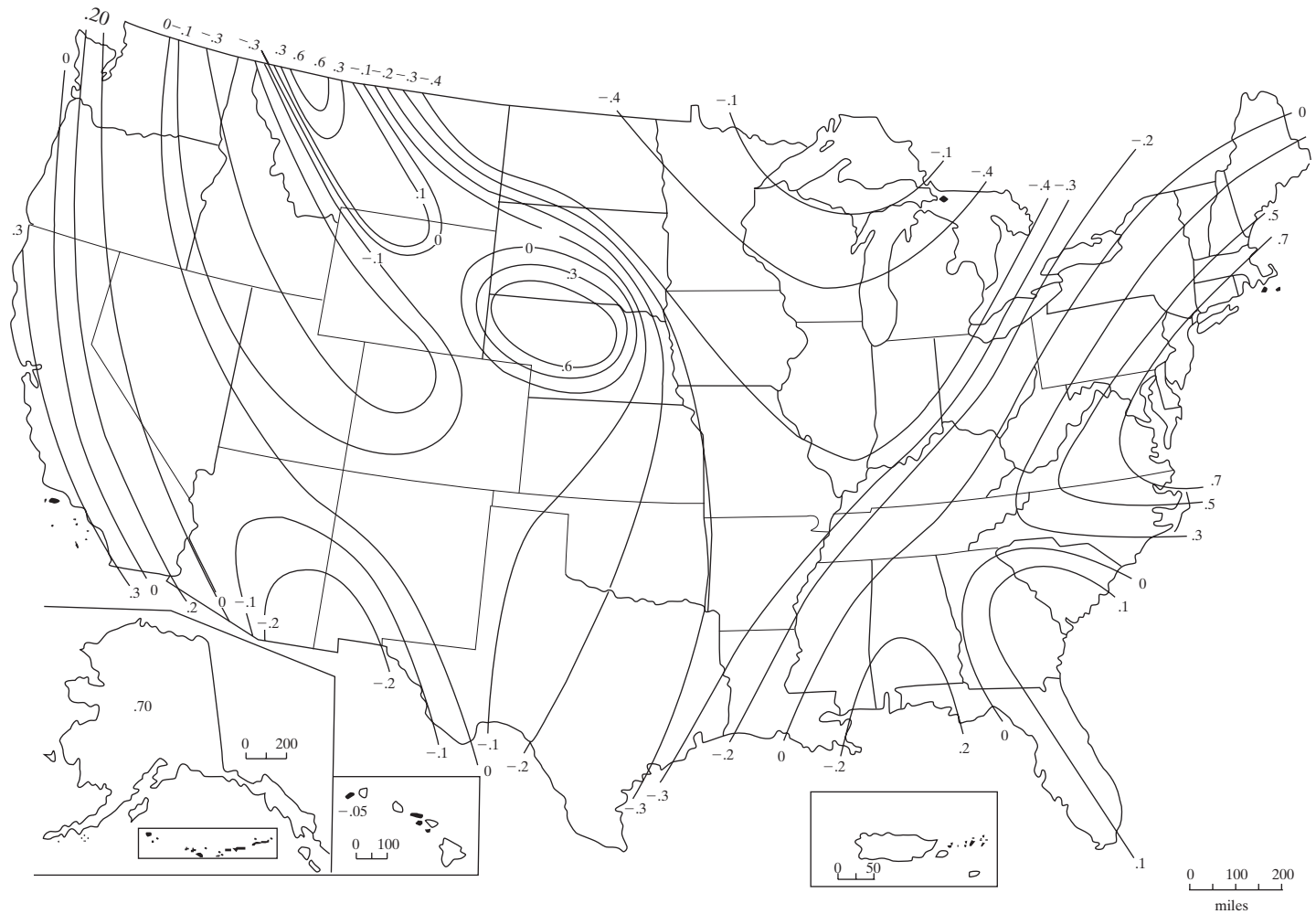


Figure 3-10

Generalized skew coefficients of logarithms of annual maximum streamflow. (From IACWD, 1982.)

in which $|C_s|$ is the absolute value of the station skew (used as an estimate of population skew) and n is the record length in years. Stedinger and Griffis (2008) describe issues with the Bulletin 17B method that may be addressed in the future. One of the more important such issue is an eventual updating of the regional skewness map and map variance (0.302); the latter is likely too high in light of available longer station records since the publication of Bulletin 17B. Griffis and Stedinger (2009) present methods for improved estimates of weighted skew that are able to incorporate updated regional skew values.

Urbanizing watersheds pose difficult problems in the estimation of moments, since the runoff tends to increase with time. Hence, the frequency distributions and moments may be nonstationary (time varying). Simulation models, described in Chapters 5 and 6, may be used to develop frequency distributions for flows in rapidly urbanizing basins at a given stage of development.

MOMENTS OF AN ANNUAL MAXIMUM SERIES

EXAMPLE 3-3

The series of 75 annual maximum flows for the Siletz River is shown in Table 3-1. Evaluate the mean and standard deviation of the original data and of the logs (base 10) of the data using Equations (3-37) and (3-38). Compare the various skewness estimates.

This exercise is easily performed in a spreadsheet. For example, Excel functions to perform the moment calculations for the column of data in Table 3-1 are shown below. Moments for $\log_{10}$ DATA are performed on the $\log_{10}$ transformation of the column of flows. Note, for instance, that the log of the mean flow is not equal to the mean of the logs; that is, $\log(20452) = 4.3107 \neq 4.2921$. (Ample significant figures should be carried when working with logarithms.) The results are presented in Table 3-2.

SOLUTION

A coefficient of variation of the flow data of 30% indicates wide variability of the flows, as is evident in Figure 3-2. Considering the $\log_{10}$ values, using the regional data from Figure 3-10 gives $C_m = 0.0$ for the

Table 3-2 Computed Moments for Annual Maximum Floods for the Siletz River, Near Siletz, Oregon, Water Years 1925-1999.

	Excel Function	Original Data (cfs)	$\log_{10}$ Data (log cfs)
Number of data points	COUNT	75	75
Mean [Eq. (3-37)]	AVERAGE	20,452 cfs	4.2921
Variance [Eq. (3-38)]	VAR	37,079,690 cfs ²	0.01665
Standard deviation	STDEV	6089 cfs	0.12905
Coefficient of variation		0.298	0.030
Skewness [Eq. (3-40)]	SKEW	0.7889	-0.1565
Weighted skewness [Eq. (3-41)]			-0.1242

north-central coastline of Oregon. A weighted average using Equation (3-41) gives an alternative estimate for the skewness of the logs of -0.1242 , somewhat less in magnitude than the station value given by Equation (3-40). Which value is more nearly correct could be determined from a study of other stations in the region; the practical effect of the small difference in this case is minor. For purposes of examples in this chapter, the weighted value of -0.1242 will be used [Eq. (3-41)]. The weighted average skewness is computed as follows:

For the Siletz River data for Oregon, using Equation (3-41), $C_m = 0.0$ and $|C_s| = 0.1565$. Therefore,

$$A = -0.33 + 0.08(0.1565) = -0.31748$$

$$B = 0.94 - 0.26(0.1565) = 0.899315$$

$$V(C_s) = 10^{A-B \log_{10}(n/10)} = 0.0786 \quad \text{and} \quad V(C_m) = 0.302 \quad \text{for the map.}$$

Finally,

$$W = \frac{0.302}{0.0786 + 0.302} = 0.793 \quad \text{and} \quad 1 - W = 0.207$$

and

$$C_w = 0.793(-0.1565) + (0.207)(0.0) = -0.1242.$$

Fitting a Distribution to Data

An intuitive use for moment estimates is to fit probability distributions to data by equating the moment estimates obtained from the data to the functional form for the distribution. For example, the normal distribution has parameters mean μ and variance σ^2 . The **method of moments** fit for the normal distribution is simply to use the estimates from the data for the mean and variance. As another example, the parameter λ of the exponential distribution is equal to the reciprocal of the mean of the distribution. Hence, the method of moments fit is $\hat{\lambda} = 1/\hat{\mu}$.

Graphical methods and the **method of maximum likelihood** are two alternative procedures to the method of moments for fitting distributions to data. Graphical methods will be discussed in Section 3.6. Although the method of maximum likelihood is superior to the method of moments by some statistical measures, it is generally much more computationally complex and is beyond the scope of this text. Maximum likelihood methods for several distributions used in hydrology are presented by Kite (1977). Graphical methods will be illustrated later.

Finally, alternatives exist to use of the product-moment estimates given in this section for variance [Eq. (3-38)] and skewness [Eq. (3-40)]. So-called **L-moments** (L for “linear”) are linear combinations of the ordered

(ranked) samples of a variable (Hosking and Wallis, 1997). Among other advantages, because no powers (“products”) of samples are used in the computation, L-moments are less subject to bias in estimation. However, they are not a part of the Bulletin 17-B (IACWD, 1982) procedure and will not be described herein. Applications to parameter estimation are shown by Stedinger et al. (1993) and Hosking and Wallis (1997).

The main objective of determining a distribution’s parameters is usually to evaluate its CDF. In some cases, however, the same purpose may be accomplished without calculating the actual distribution parameters. Instead, the distribution is evaluated using a frequency factor K , defined as

$$K = \frac{x - \bar{x}}{S_x}. \quad (3-43)$$

The value of K is a function of the desired value for the CDF and may also be a function of the skewness. Hence, if $K(F, C_s)$ is known for the calculated skewness and desired CDF value, the corresponding magnitude of x can be calculated. Frequency factors will be illustrated subsequently for several of the distributions to be discussed.

The most common means used in hydrology to indicate the probability of an event is to assign a **return period**, or **recurrence interval**, to the event. The return period is defined as follows:

3.4 RETURN PERIOD OR RECURRENCE INTERVAL

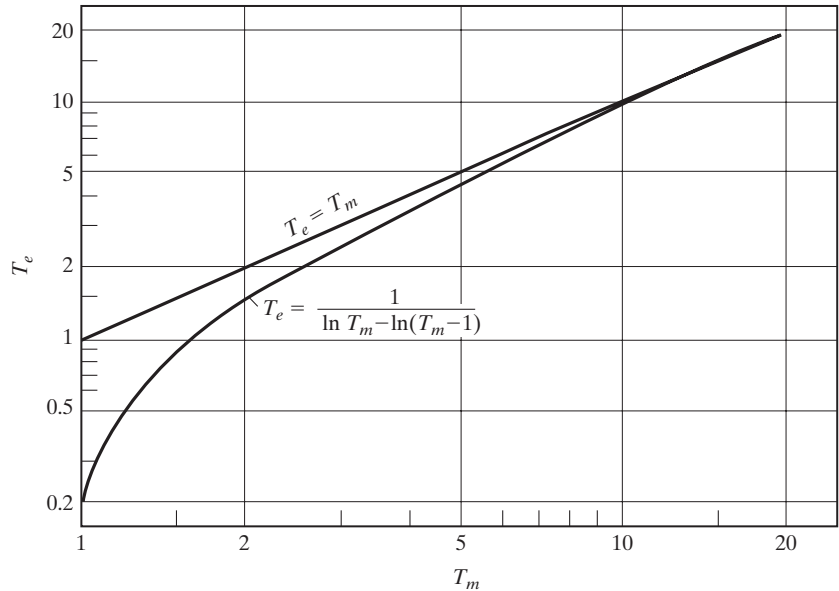
An annual maximum event has a return period (or recurrence interval) of T years if its magnitude is equaled or exceeded once, *on the average*, every T years. The reciprocal of T is the exceedance probability, $1 - F$, of the event—that is, the probability that the event is equaled or exceeded in any one year.

Thus, the 50-yr flood has a probability of 0.02, or 2%, of being equaled or exceeded in any single year. It is imperative to realize that the return period implies nothing about the actual time sequence of an event. The 50-yr flood does not occur like clockwork once every 50 yr. Rather, one expects that on the average, about twenty 50-yr floods will be experienced during a 1000-yr period. There could in fact be two 50-yr floods in a row (with probability $0.02 \times 0.02 = 0.0004$ for independent events). For this reason, labeling a 50-yr flood as a “2% flood” is often a clearer indication to the public as to its true likelihood of occurrence in any one year.

The concept of a return period implies independent events and is usually found by analyzing the series of maximum annual floods (or rainfalls, etc.). The largest event in one year is assumed to be independent of the largest event in any other year. But it is also possible to apply such an analysis to the n largest independent events from an n -yr period, regardless of the year in which they occur. In this case, if the second largest event in one year was greater than the largest event in another year, it could be included in the

Figure 3-11

Relationship between return period for annual exceedances T_e and return period for annual maxima T_m [Eq. (3-44)].



frequency analysis. This series of n largest (independent) values is called the series of **annual exceedances**, as opposed to an annual maximum series. Both series are used in hydrology, with little difference at high return periods (rare events). There are likely to be more problems of ensuring independence when using annual exceedances, but for low return periods annual exceedances give a more realistic lower return period for the same magnitude than do annual maxima. The relationship between return period based on annual exceedances T_e and annual maxima T_m is (Chow, 1964)

$$T_e = \frac{1}{\ln T_m - \ln(T_m - 1)}. \quad (3-44)$$

This relationship is shown in Figure 3-11.

EXAMPLE 3-4

ANNUAL EXCEEDANCES AND ANNUAL MAXIMA

Consider the following hypothetical sequence of river flows.

Year	Three Highest Independent Flows (cfs)		
1	700	300	150
2	900	600	100
3	550	400	200
4	850	650	350
5	500	350	100

The sequences of ranked annual maxima and annual exceedances are as follows:

Rank	Annual Maxima	Annual Exceedances
1	900	900
2	850	850
3	700	700
4	550	650
5	500	600

Return periods may be assigned to minimum events (e.g., droughts) in an entirely analogous manner, with the interpretation that “equaled or exceeded” means “equal to or more severe than.” Thus, a 20-yr low flow is one for which the probability, F , is 5% that the flow will be less than or equal to the given value in any one year.

Return periods need not be limited to units of years. As long as the events are independent, months or even weeks can be used. The 6-month rainfall thus has a probability of 1/6 of being equaled or exceeded in any one month.

A further option is to use all the data in the historic time series, whether or not they are independent. This is called the **complete series**; an example is the series of 365 daily average streamflows. Note that the maximum daily average streamflow is not necessarily the maximum flow for the day, since the U.S. Geological Survey records hourly or 15-min values and determines the maximum flow as the highest recorded value and the daily average as the average of the 24 or 96 recorded values. Frequency information derived from such a complete series is usually shown in a **flow-duration curve** (Searcy, 1959), which is a plot of magnitude vs. percentage of time the magnitude is equaled or exceeded (Fig. 3-12). The information from such an analysis cannot be directly related to return period, since the values in the complete series are not necessarily independent; for example, day-to-day average streamflow values are highly correlated. Flow-duration curves are especially useful for hydropower development or any water use that depends upon a reliable minimum flow (Warnick, 1984; Mays, 2001).

FLOW DURATION CURVE

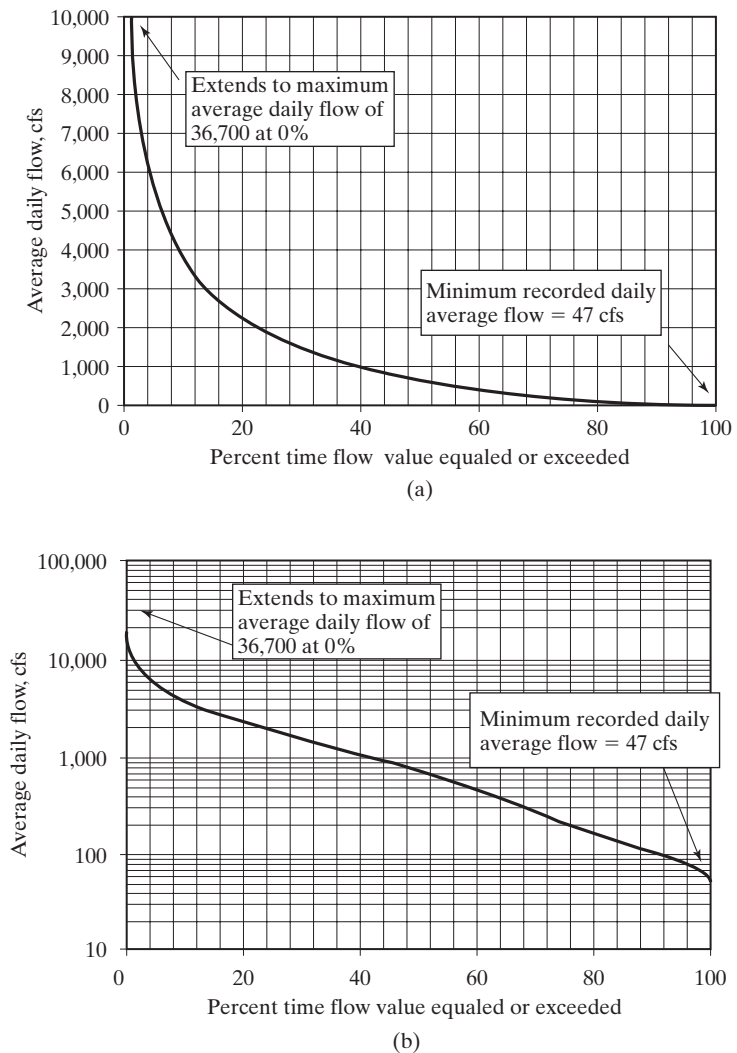
EXAMPLE 3-5

A flow-duration curve was developed for the Siletz River using 75 years (water years 1925–1999) of daily average flow values. The values were sorted on a spreadsheet and cumulative percentages assigned based on their ranking. (This can automatically be performed by Excel using the Rank and Percentile analysis tool.) Two views are shown in Figure 3-12; the arithmetic scale is used in studies of firm yield, whereas the log scale is useful in order to view the entire range of flows. The Siletz River is uncontrolled

(no dams or significant diversions above the gaging point), so low flows are not modified (raised) by any upstream storage, nor are high flows reduced. Storage would be required to raise the firm yield of the river above the very low minimum flow of 47 cfs. The area under the flow-duration curve (with arithmetic scales) is the average daily flow (also obtainable from a simple average of the daily values), and the median daily flow is the 50% value. For the Siletz River, these values are 1510 and 737 cfs, respectively, a reflection of many low flows during dry Oregon summers and “peaky” runoff during winter storms. Note that the maximum average daily flow (36,700 cfs) is less than the instantaneous hourly peak flow (40,500 cfs, Table 3-1), but both were recorded on the same day, December 28, 1998.

Figure 3-12

Flow-duration curve for the Siletz River. (a) Arithmetic scale, used for analysis of yield for water supply. (b) Logarithmic scale, useful when maximum and minimum flows have large separation.



References and Objectives

3.5 COMMON PROBABILISTIC MODELS

Many discrete PMFs and continuous PDFs are used in hydrology, but this text can focus on only a few of the most common. Others are covered in references on statistical hydrology, such as Gumbel (1958), Chow (1964), Benjamin and Cornell (1970), Kite (1977), Stedinger et al. (1993), and Haan (2002). It is especially difficult to pare down the number of continuous distributions for inclusion, since at least 10 could be selected that have been applied to flood flows. However, the most common are the normal, lognormal, gamma (Pearson type 3), and log-gamma (log Pearson type 3); only these four are included here. In addition, the exponential distribution is discussed because of its simplicity and its application to interevent times.

The objective of a discrete analysis is most often to assign probabilities to the number of occurrences of an event, whereas the objective of a continuous analysis is most often to determine the probability of the magnitude of an event, and vice versa. For the discrete analysis, there may be interest in both the PMF and the CDF, but for continuous analysis the value of the PDF itself is rarely of interest. Rather, only the CDF for the continuous random variable need be evaluated. These distinctions will be seen as the various distributions are presented.

The Binomial Distribution

It is common to examine a sequence of independent events for which the outcome of each can be either a success or a failure; for example, either the T -yr flood occurs or it does not. Such a sequence consists of Bernoulli trials, independent trials for which the probability of success at each trial is a constant p . The binomial distribution answers the question: What is the probability of exactly x successes in n Bernoulli trials? This will be the only discrete distribution considered in this text; it is very commonly used in hydrology.

The probability that there will be x successes followed by $n - x$ failures is just the product of the probabilities of the n independent events: $p^x(1 - p)^{n-x}$. But this represents just one possible sequence for x successes and $n - x$ failures; all possible sequences must be considered, including those in which the successes do not occur consecutively. The number of possible ways (combinations) of choosing x events out of n possible events is given by the **binomial coefficient** (Parzen, 1960):

$$\binom{n}{x} = \frac{n!}{x!(n-x)!}. \quad (3-45)$$

Thus, the desired probability is the product of the probability of any one sequence and the number of ways in which such a sequence can occur:

$$p(x) = \binom{n}{x} p^x (1 - p)^{n-x}, \quad x = 0, 1, 2, 3, \dots, n. \quad (3-46)$$

The notation $B(n, p)$ indicates the binomial distribution with parameters n and p ; example PMFs are shown in Figure 3–13. From Equations (3–19) and (3–28), the mean and variance of x are

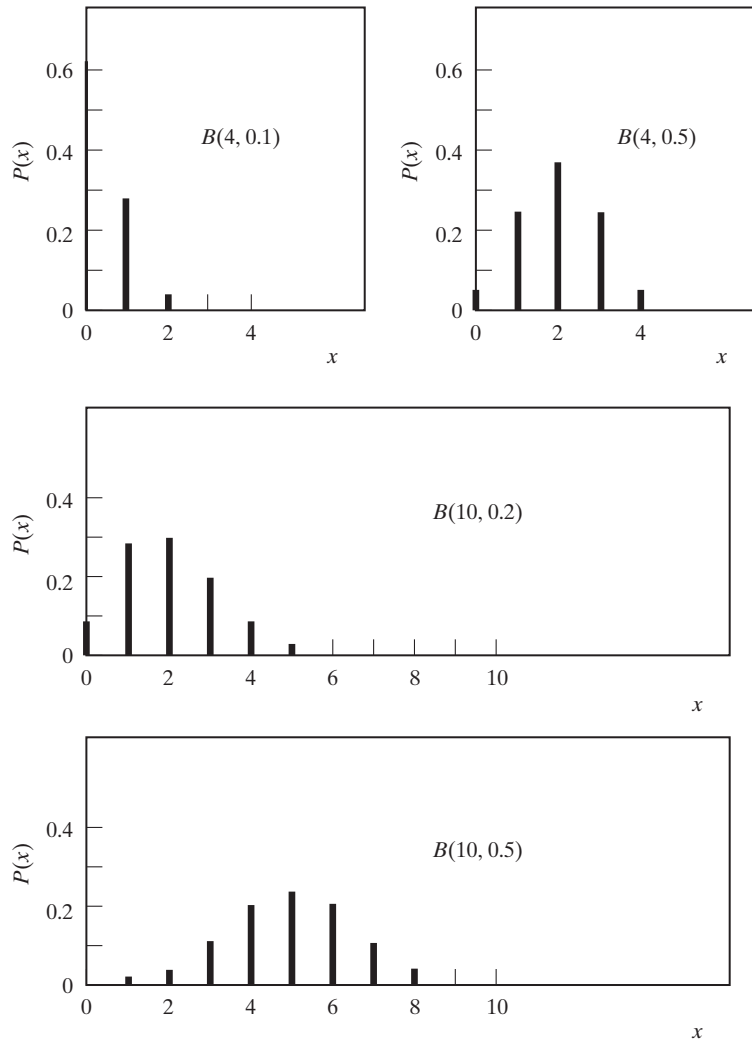
$$E(x) = np, \quad (3-47)$$

$$\text{Var}(x) = np(1 - p). \quad (3-48)$$

The skewness is

$$g = \frac{1 - 2p}{[np(1 - p)]^{0.5}}. \quad (3-49)$$

Figure 3–13
Binomial probability
mass function (PMF).



Clearly, the skewness is zero and the distribution is symmetric if $p = 0.5$.

The cumulative distribution function is

$$F(x) = \sum_{i=0}^x \binom{n}{i} p^i (1-p)^{n-i}. \quad (3-50)$$

Evaluation of the CDF can get very cumbersome for large values of n and intermediate values of x . It is tabulated by the Chemical Rubber Company (n.d.) and by the National Bureau of Standards (1950). For large values of n , the relationship between the binomial and beta distribution may be used (Abramowitz and Stegun, 1964; Benjamin and Cornell, 1970), or it may be approximated by the normal when $p \approx 0.5$. However, the easiest option is to use available functions in spreadsheets. For example, the function BINOMDIST in Excel will evaluate both the PMF and CDF for any desired x , n , and p .

RISK AND RELIABILITY

EXAMPLE 3-6

Over a sequence of n -yr, what is the probability that the T -yr event will occur at least once? The probability of occurrence in any one year (event) is $p = 1/T$, and the probability of x occurrences in n years is $B(n, p)$. The Prob (at least one occurrence in n events) is called the **risk**. Thus, the risk is the sum of the probabilities of one flood, two floods, three floods, . . . , n floods occurring during the n -yr period, but this would be a very tiresome way in which to compute it. Instead,

$$\begin{aligned} \text{Risk} &= 1 - P(0) \\ &= 1 - \text{Prob}(\text{no occurrence in } n \text{ yr}) \\ &= 1 - (1 - p)^n \\ &= 1 - (1 - 1/T)^n. \end{aligned} \quad (3-51)$$

Reliability is defined as $1 - \text{risk}$. Thus,

$$\begin{aligned} \text{Reliability} &= (1 - p)^n \\ &= (1 - 1/T)^n. \end{aligned} \quad (3-52)$$

This concept of risk and reliability is very important for hydrologic design. Equation (3-51) can be used to determine the return period required for a given design life and level of risk. Values are shown in Table 3-3 that illustrate the very high return periods required for low risk for a long design life.

Table 3-3 Return Periods for Various Degrees of Risk and Expected Design Life [Eq. (3-51)]

Risk (%)	Reliability (%)	Expected Design Life, n (yr)							
		2	5	10	15	20	25	50	100
75	25	2.0	4.1	7.7	11.3	14.9	18.5	36.6	72.6
63	37	2.6	5.5	10.6	15.6	20.6	25.6	50.8	101.1
50	50	3.4	7.7	14.9	22.1	29.4	36.6	72.6	144.8
40	60	4.4	10.3	20.1	29.9	39.7	49.4	98.4	196.3
30	70	6.1	14.5	28.5	42.6	56.6	70.6	140.7	280.9
25	75	7.5	17.9	35.3	52.6	70.0	87.4	174.3	348.1
20	80	9.5	22.9	45.3	67.7	90.1	112.5	224.6	448.6
15	85	12.8	31.3	62.0	92.8	123.6	154.3	308.2	615.8
10	90	19.5	48.0	95.4	142.9	190.3	237.8	475.1	949.6
5	95	39.5	98.0	195.5	292.9	390.4	487.9	975.3	1950.1
2	98	99.5	248.0	495.5	743.0	990.5	1238.0	2475.4	4950.3
1	99	199.5	498.0	995.5	1493.0	1990.5	2488.0	4975.5	9950.4
0.5	99.5	399.5	998.0	1995.5	2993.0	3990.5	4988.0	9975.5	19950.5

EXAMPLE 3-7**CRITICAL FLOOD DESIGN**

Consider the 50-yr flood ($p = 0.02$).

- (a) What is the probability that at least one 50-yr flood will occur during the 30-yr lifetime of a flood control project? This is just the risk of failure discussed above, and the distribution of the number of failures is $B(30, 0.02)$. Thus, from Equation (3-51),

$$\begin{aligned}
 \text{Risk} &= 1 - (1.0 - 0.02)^{30} \\
 &= 1 - 0.98^{30} \\
 &= 1 - 0.545 \\
 &= 0.455.
 \end{aligned}$$

If this risk is too great, the engineer might design for the 100-yr event for which the risk is

$$\text{Risk} = 1 - 0.99^{30} = 0.26$$

and the reliability is 0.74. For this latter circumstance, there is a 26% chance of occurrence of the 100-yr event over the 30-yr lifetime of the project.

When it is not possible to reduce the risk to a desired level by designing for a high (but hypothetical) return period event, an alternative is to design for the **probable maximum flood**, which is the flood that results from the **probable maximum precipitation (PMP)** event. The PMP is the highest precipitation likely to occur under known meteorological conditions (Smith, 1993; Mays, 2001)

and has been computed for most areas of the United States by the National Weather Service Hydrometeorological Design Studies Center (<http://www.nws.noaa.gov/oh/hdsc/studies/pmp.html>).

- (b) What is the probability that the 100-yr flood will not occur in 10 yr? In 100 yr?

$$\text{For } n = 10, \quad P(x = 0) = (1 - p)^{10} = 0.99^{10} = 0.904,$$

$$\text{For } n = 100, \quad P(x = 0) = (1 - p)^{100} = 0.99^{100} = 0.37.$$

Thus, there is a 37% chance (37% reliability) that the 100-yr flood will not occur during a sequence of 100 yr.

- (c) In general, what is the probability of having no floods greater than the T -yr flood during a sequence of T yr?

$$P(x = 0) = (1 - 1/T)^T.$$

As T gets large, this approaches $1/e$ (Benjamin and Cornell, 1970, p. 234). Thus for large T ,

$$P(x = 0) \approx e^{-1} = 0.368 \quad (\text{the reliability})$$

and

$$P(x \geq 0) \approx 1 - e^{-1} = 0.632 \quad (\text{the risk}).$$

As an approximation, the T -yr flood will occur within a T -yr period with approximately a $2/3$ probability.

COFFERDAM DESIGN

EXAMPLE 3-8

A cofferdam has been built to protect homes in a floodplain until a major channel project can be completed. The cofferdam was built for the 20-yr flood event. The channel project will require 3 yr to complete. Hence, the process is $B(3, 0.05)$. What are the probabilities that

- (a) The cofferdam will not be overtopped during the 3 yr (the reliability)?

$$\text{Reliability} = (1 - 1/20)^3 = 0.95^3 = 0.86.$$

- (b) The cofferdam will be overtopped in any one year?

$$\text{Prob} = 1/T = 0.05.$$

- (c) The cofferdam will be overtopped exactly once in 3 yr?

$$P(x = 1) = \binom{3}{1} p^1 (1 - p)^2 = 3 \cdot 0.05 \cdot 0.95^2 = 0.135.$$

(d) The cofferdam will be overtopped at least once in 3 yr (the risk)?

$$\text{Risk} = 1 - \text{Reliability} = 0.14.$$

(e) The cofferdam will be overtopped only in the third year?

$$\text{Prob} = (1 - p)(1 - p)p = 0.95^2 \cdot 0.05 = 0.045.$$

The Exponential Distribution

Consider a process of random arrivals such that the arrivals (events) are independent, the process is stationary (the parameters of the process do not change with time), and it is not possible to have more than one arrival at an instant in time. These conditions describe a Poisson process (Benjamin and Cornell, 1970), which is often representative of the arrival of storm events. If the random variable t represents the interarrival time (the time between events), it is found to be exponentially distributed with PDF

$$f(t) = \lambda e^{-\lambda t}, \quad t \geq 0. \quad (3-53)$$

This PDF is sketched in Figure 3-14.

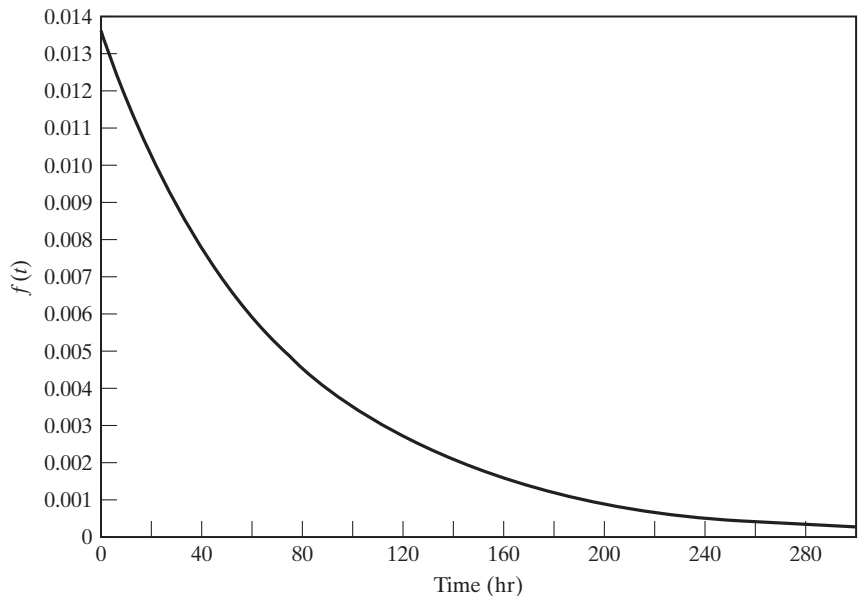
The mean of the distribution is

$$E(t) = 1/\lambda. \quad (3-54)$$

and the variance is

$$\text{Var}(t) = 1/\lambda^2. \quad (3-55)$$

Figure 3-14
Exponential PDF
parameters for
Example 3-9.



Thus, this distribution has the interesting property that its mean equals its standard deviation, or $CV = 1.0$, where CV is the coefficient of variation. The distribution is obviously skewed to the right, with a constant skewness, $g = 2$.

The CDF may easily be evaluated analytically:

$$F(t) = \int_0^t \lambda e^{-\lambda\tau} d\tau = 1 - e^{-\lambda t}. \quad (3-56)$$

Clearly, when $t = \infty$, $F(\infty) = 1$, as it should (the area under the PDF must equal 1).

The exponential distribution may be easily manipulated analytically and as a result is sometimes used to approximate more complex skewed distributions, such as the gamma or extreme value. It is occasionally used to fit rainfall totals or river flows but is most often applied to interevent times. Closely related distributions are the Poisson, which is the PMF for the number of arrivals in time t , and the gamma, which is the PDF for the time between k events (Benjamin and Cornell, 1970).

STORM INTERARRIVAL TIMES

EXAMPLE 3-9

During the course of a year, about 110 independent storm events occur at Gainesville, Florida, and their average duration is 5.3 hr. Ignoring seasonal variations, in a year of 8760 hr, the average interevent time is

$$\bar{t} = \frac{8760 - 110 \cdot 5.3}{110} = 74.3 \text{ hr.}$$

The exponential distribution may thus be fit by the method of moments with $\hat{\lambda} = 1/\bar{t} = 0.0135 \text{ hr}^{-1}$.

- (a) What is the probability that at least 4 days = 96 hr elapse between storms?

$$\text{Prob}(t \geq 96) = 1 - F(96) = e^{-0.0135 \cdot 96} = 0.27.$$

- (b) What is the probability that the separation between two storms will be exactly 12 hr?

$$\text{Prob}(t = 12) = 0,$$

since the probability that a continuous variable exactly equals a specific value is zero.

- (c) What is the probability that the separation between two storms will be less than or equal to 12 hr?

$$\text{Prob}(t \leq 12) = F(12) = 1 - e^{-0.0135 \cdot 12} = 1 - 0.85 = 0.15.$$

The Normal Distribution

The normal distribution is also known as the Gaussian distribution, or normal error curve, and is fundamental to the entire realm of probability and statistics. One reason is that the central limit theorem states that under very general conditions, as the number of variables in the sum becomes large, the distribution of the sum of a large number of random variables will approach the normal distribution, regardless of the underlying distribution (Benjamin and Cornell, 1970). Many physical processes can be conceptualized as the sum of individual processes. Hence, the normal distribution finds many applications in hydrology as well as in the statistical areas of hypothesis testing, confidence intervals, and quality control.

The PDF for the normal (the bell-shaped curve) is included with three other PDFs in Figure 3-15 and is given by

$$f(x) = \frac{1}{\sqrt{2\pi}\sigma} e^{-(1/2)[(x-\mu)/\sigma]^2}, \quad -\infty \leq x \leq \infty. \quad (3-57)$$

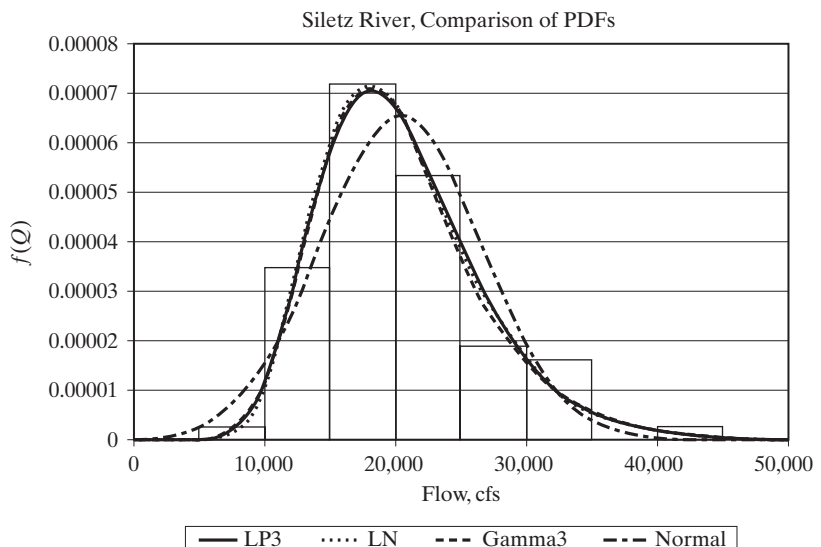
The parameters of the distribution are the mean μ and variance σ^2 themselves, and the skewness is zero. The method-of-moments fit is therefore simply to use the estimates $\bar{x}$ and S_x^2 in the distribution. Random variables having the distribution of Equation (3-57) are denoted by $N(\mu, \sigma^2)$. Although the symmetric nature of the normal distribution usually makes it unsuitable for flood flows (or other extremes), it often describes annual totals (e.g., annual runoff in ac-ft) quite well.

The CDF is evaluated after a change of variable to

$$z \equiv (x - \mu)/\sigma, \quad (3-58)$$

Figure 3-15

Four PDFs fit to data for the Siletz River. Fit is by the method of moments, as shown in the text, with moments given in Example 3-3.



where z is known as the standard normal variate and is $N(0, 1)$. The CDF is then

$$F(z) = \int_{-\infty}^z \frac{1}{\sqrt{2\pi}} e^{-u^2/2} du. \quad (3-59)$$

Unfortunately, the integration cannot be performed analytically, but tables of $F(z)$ vs. z are found in virtually every statistics text and in this book in Appendix D. However, the process may be greatly simplified through the use of spreadsheets. For example, the Excel functions NORMDIST and NORMINV may be used to obtain the cumulative, $F[(x - \bar{x})/S_x]$, and the inverse, $x = F^{-1}(\bar{x}, S_x, F)$, respectively.

The similarity of Equation (3-58) to Equation (3-43) for the frequency factor is not accidental. In fact, z corresponds exactly to the frequency factor K , discussed in Section 3.3. The magnitude of x for a given return period can be found easily by the following procedure:

1. $F(z) = 1 - 1/T$.
2. Obtain z from Appendix D.
3. Then $x = \bar{x} + z \cdot S_x$.

APPLICATION OF NORMAL DISTRIBUTION TO FLOOD FLOWS

EXAMPLE 3-10

The normal distribution is to be fit to the data for the Siletz River of Table 3-2 and Example 3-3. The fit is not expected to be very good, since the data are so skewed.

- (a) What is the 100-yr flood?

$$F(Q_{100}) = 1 - 1/T = 0.99.$$

From Table D-2, $z = 2.326$. Thus,

$$Q_{100} = \bar{Q} + z \cdot S_Q = 20,452 + 2.326 \cdot 6089 = 34,620 \text{ cfs.}$$

- (b) What is the probability that a flood will be less than or equal to 30,000 cfs?

$$z = \frac{30,000 - 20,452}{6089} = 1.568.$$

By linear interpolation in Table D-1,

$$F(1.568) = 0.9416 = \text{Prob}(Q \leq 30,000).$$

(For comparison, the Excel function NORMDIST returns $F = 0.941551$.)

The return period of a flow of 30,000 cfs is thus

$$T = \frac{1}{1 - F} = 17 \text{ yr.}$$

The Lognormal Distribution

Consider a hypothetical runoff calculation, in which runoff is equal to the *product* of functions of several random factors, such as rainfall, contributing area, loss coefficient, and evaporation. In general, if a random variable x results from the product of a large number of other random variables (i.e., a “multiplicative mechanism”), then the distribution of the logarithm of x will approach normality, since the logarithm of x will consist of the sum of the logs of the contributing factors. In hydrology, it is easy to conceive of many contributing factors to runoff that are inherently random and about which there is very little deterministic information. Hence, a multiplicative mechanism for runoff may be a very reasonable assumption. In general, a random variable has a **lognormal** distribution if the log of the random variable is distributed normally. For example, if $y = \log x$ and if y is distributed normally, then x is distributed lognormally.

The lognormal PDF is sketched in Figure 3–15. Because it is bounded on the left by zero, has a positive skewness, and is easily used through its relationship to the normal, it is widely applied in hydrology. The skewness is a function of the coefficient of variation:

$$g = 3CV + CV^3. \quad (3-60)$$

The skewness of the transformed variable y is, of course, zero, since y is distributed normally.

Although the PDF of x (the untransformed variable) can easily be derived (Benjamin and Cornell, 1970), it is seldom needed. Instead, the moments of $y = \log x$ (either base 10 or natural logs may be used) are found, and the normal distribution is used for y . This may be done in two ways. The method of moments requires that the moments of the untransformed data be calculated and related to the moments of the logs. When the natural log is used (i.e., $y = \ln x$), these relationships are

$$CV_x^2 \equiv \frac{\sigma_x^2}{\mu_x^2} = e^{\sigma_y^2} - 1, \quad (3-61)$$

and

$$\mu_x = e^{\mu_y + \sigma_y^2/2}, \quad (3-62)$$

which can be solved for μ_y and σ_y^2 . (If base 10 logarithms are used, the mean and standard deviation must be converted to those for natural logs by multiplying by $\ln 10 = 2.3026$ prior to the use of Equations (3–61) and (3–62); see Example 3–11, part (c)). The method of moments using Equations (3–61) and (3–62) will preserve the moments of x , should it be necessary (e.g., for simulation of river flows; see Fiering and Jackson, 1971, and Section 3.7).

Note that the log of the mean is *not* the mean of the logs. Rather,

$$\mu_y = \log x_m, \quad (3-63)$$

where x_m is the median of x . Another useful relationship of the lognormal distribution is

$$\mu_x = x_m(1 + CV_x^2), \quad (3-64)$$

which demonstrates again that the mean value of x is greater than the median (see Fig. 3-9).

Estimation of the parameters directly from the log-transformed data is more common and amounts to the maximum likelihood estimates for the parameters of y . In this case, logarithms of all the flows are calculated first. Then the mean and variance of the log-transformed data are used. If moments of the transformed data are calculated by both methods, their correspondence will increase as the sample size increases.

The three-parameter lognormal distribution has an additional parameter that is a nonzero lower bound on the value of x . Three moments are required to estimate the three parameters; applications to hydrology are given by Sangal and Biswas (1970).

APPLICATION OF LOGNORMAL DISTRIBUTION TO FLOOD FLOWS

EXAMPLE 3-11

Repeat Example 3-10 using the lognormal distribution. Parameters of the logs (base 10) were calculated in Example 3-3.

- (a) What is the 100-yr flood?

The z value was found in Example 3-10 to be 2.326. Thus,

$$y_{100} = 4.29209 + 2.326 \cdot 0.12905 = 4.5923$$

and

$$Q_{100} = 10^{y_{100}} = 39,100 \text{ cfs.}$$

This value is much higher than the estimate (34,620 cfs) from the normal distribution. But it is a better estimate because the lognormal distribution more accurately reflects the skewness of the actual flow data. (Note that it is a good practice to carry several significant figures when working with logarithms, and round only for the final answer.)

- (b) What is the probability that a flow will be less than or equal to 30,000 cfs?

The value of $y = \log_{10}(30,000) = 4.4771$.

$$z = \frac{4.4771 - 4.2921}{0.12905} = 1.4337.$$

Interpolating linearly in Table D-1, $F(1.4337) = 0.9242$. The return period of a 30,000-cfs flood is thus $1/(1 - 0.9242) = 13.1$ yr.

- (c) How do the moment estimates from the log-transformed data compare with those found using Equations (3-61) and (3-62)? These two equations must be solved for the moments of the logs. With data from Table 3-2, using symbols for the estimated mean and variance, and rearranging the equations gives values for natural logs:

$$S_y^2 = \ln(CV_x^2 + 1) = \ln(0.298^2 + 1) = 0.0849 \quad (3-65)$$

and

$$\bar{y} = \ln \bar{x} - S_y^2/2 = \ln 20452 - 0.0849/2 = 9.8834. \quad (3-66)$$

Converting to base 10 logs by dividing the mean by 2.3026 and the variance by 2.3026^2 gives the following results.

	Using Moment Relationships	Using Transformed Data
$\bar{y}$	4.292	4.292
S_y^2	0.0175	0.0167
S_y	0.132	0.129

The values are unusually close for this example, but that is not always the case, especially for shorter records. However, use of moments obtained directly from the transformed data is most common, easiest, and preferable.

The Gamma (Pearson Type 3) Distribution

This distribution receives extensive use in hydrology simply because of its shape and its well-known mathematical properties. The three-parameter gamma distribution is sketched in Figure 3-15 and has the pleasing properties of being bounded on the left (the lower limit is 5015 cfs) and of positive skewness. (The distribution can also be used with negative skewness.) Although the three parameters of the PDF are simple functions of the mean, variance, and skewness (Kite, 1977; Bobée and Ashkar, 1991), it is more common in hydrologic applications to evaluate the CDF with frequency factors (Section 3.3), for which the PDF parameters are unnecessary. (However, note that spreadsheet functions such as Excel's GAMMADIST and GAMMAINV make it possible to avoid the tedious process of using any tabular values; see Problem 3.12.) The frequency factors K are a function of the skewness and return period (or CDF) and are given in Table 3-4. Thus, to evaluate the T -yr flood, the moments of the data are computed and

$$Q_T = \bar{Q} + K(C_s, T) \cdot S_Q. \quad (3-67)$$

This procedure is satisfactory for return periods listed in Table 3-4, but interpolation between return periods is not appropriate. Instead, the

Table 3-4 Frequency Factors K for Gamma and Log Pearson Type 3 Distributions

Skew Coefficient C_s	Recurrence Interval in Years							
	1.0101	2	5	10	25	50	100	200
	Percent Chance ($\geq$) = $1 - F$							
	99	50	20	10	4	2	1	0.5
3.0	-0.667	-0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	-0.690	-0.390	0.440	1.195	2.277	3.134	4.013	4.904
2.8	-0.714	-0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	-0.740	-0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	-0.769	-0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	-0.799	-0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	-0.832	-0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	-0.867	-0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	-0.905	-0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	-0.946	-0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	-0.990	-0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	-1.037	-0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	-1.087	-0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	-1.140	-0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	-1.197	-0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	-1.256	-0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	-1.318	-0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	-1.383	-0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	-1.449	-0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	-1.518	-0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	-1.588	-0.164	0.758	1.340	2.043	2.542	3.022	3.489
.9	-1.660	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
.8	-1.733	-0.132	0.780	1.336	1.993	2.453	2.891	3.312
.7	-1.806	-0.116	0.790	1.333	1.967	2.407	2.824	3.223
.6	-1.880	-0.099	0.800	1.328	1.939	2.359	2.755	3.132
.5	-1.955	-0.083	0.808	1.323	1.910	2.311	2.686	3.041
.4	-2.029	-0.066	0.816	1.317	1.880	2.261	2.615	2.949
.3	-2.104	-0.050	0.824	1.309	1.849	2.211	2.544	2.856
.2	-2.178	-0.033	0.830	1.301	1.818	2.159	2.472	2.763
.1	-2.252	-0.017	0.836	1.292	1.785	2.107	2.400	2.670
0	-2.326	0	0.842	1.282	1.751	2.054	2.326	2.576
-0.1	-2.400	0.017	0.846	1.270	1.716	2.000	2.252	2.482
-0.2	-2.472	0.033	0.850	1.258	1.680	1.945	2.178	2.388
-0.3	-2.544	0.050	0.853	1.245	1.643	1.890	2.104	2.294
-0.4	-2.615	0.066	0.855	1.231	1.606	1.834	2.029	2.201
-0.5	-2.686	0.083	0.856	1.216	1.567	1.777	1.955	2.108
-0.6	-2.755	0.099	0.857	1.200	1.528	1.720	1.880	2.016
-0.7	-2.824	0.116	0.857	1.183	1.488	1.663	1.806	1.926
-0.8	-2.891	0.132	0.856	1.166	1.448	1.606	1.733	1.837
-0.9	-2.957	0.148	0.854	1.147	1.407	1.549	1.660	1.749
-1.0	-3.022	0.164	0.852	1.128	1.366	1.492	1.588	1.664

Skew Coefficient C_s	Recurrence Interval in Years							
	1.0101	2	5	10	25	50	100	200
	Percent Chance ($\geq$) = $1 - F$							
	99	50	20	10	4	2	1	0.5
-1.1	-3.087	0.180	0.848	1.107	1.324	1.435	1.518	1.581
-1.2	-3.149	0.195	0.844	1.086	1.282	1.379	1.449	1.501
-1.3	-3.211	0.210	0.838	1.064	1.240	1.324	1.383	1.424
-1.4	-3.271	0.225	0.832	1.041	1.198	1.270	1.318	1.351
-1.5	-3.330	0.240	0.825	1.018	1.157	1.217	1.256	1.282
-1.6	-3.388	0.254	0.817	0.994	1.116	1.166	1.197	1.216
-1.7	-3.444	0.268	0.808	0.970	1.075	1.116	1.140	1.155
-1.8	-3.499	0.282	0.799	0.945	1.035	1.069	1.087	1.097
-1.9	-3.553	0.294	0.788	0.920	0.996	1.023	1.037	1.044
-2.0	-3.605	0.307	0.777	0.895	0.959	0.980	0.990	0.995
-2.1	-3.656	0.319	0.765	0.869	0.923	0.939	0.946	0.949
-2.2	-3.705	0.330	0.752	0.844	0.888	0.900	0.905	0.907
-2.3	-3.753	0.341	0.739	0.819	0.855	0.864	0.867	0.869
-2.4	-3.800	0.351	0.725	0.795	0.823	0.830	0.832	0.833
-2.5	-3.845	0.360	0.711	0.771	0.793	0.798	0.799	0.800
-2.6	-3.889	0.368	0.696	0.747	0.764	0.768	0.769	0.769
-2.7	-3.932	0.376	0.681	0.724	0.738	0.740	0.740	0.741
-2.8	-3.973	0.384	0.666	0.702	0.712	0.714	0.714	0.714
-2.9	-4.013	0.390	0.651	0.681	0.683	0.689	0.690	0.690
-3.0	-4.051	0.396	0.636	0.660	0.666	0.666	0.667	0.667

Source: IACWD (1982).

magnitudes should be plotted vs. return period or vs. the CDF, and graphical interpolation performed. (Graphical procedures will be discussed in Section 3.6.) This is also the procedure to use for the inverse problem of finding the probability (or return period) corresponding to a given flood magnitude. Alternatively, the three parameters of the distribution may be found and tables of the CDF (or spreadsheet functions) used to determine either the magnitude or the probability; see Problem 3.12. The gamma distribution is directly related to the chi-square distribution, for which many tables are available (Abramowitz and Stegun, 1964; Benjamin and Cornell, 1970; Haan, 2002).

The two-parameter gamma distribution corresponds to setting the left boundary (Fig. 3-15) to zero. In this case, the sample skewness need not be calculated. Instead, the skewness of the two-parameter gamma should be used:

$$g = 2CV \quad \text{or} \quad C_s = 2S_x/\bar{x} = 2CV. \quad (3-68)$$

This skewness can then be used in Table 3-4.

APPLICATION OF GAMMA DISTRIBUTION TO FLOOD FLOWS**EXAMPLE 3-12**

What is the magnitude of the 100-yr flood for the Siletz River ($\bar{Q} = 20,452$ cfs, $S_Q = 6089$ cfs) using the gamma-3 and gamma-2 distributions?

For the gamma-3, $C_s = 0.789$. Linear interpolation in Table 3-4 gives $K = 2.884$. Thus,

$$Q_{100} = 20,452 + 2.884 \cdot 6089 = 38,010 \text{ cfs.}$$

For the gamma-2, $C_s = 2CV = 2 \cdot 0.298 = 0.596$. Linear interpolation in Table 3-4 gives $K = 2.752$. Thus,

$$Q_{100} = 20,452 + 2.752 \cdot 6089 = 37,210 \text{ cfs.}$$

The slightly lower value for the gamma-2 reflects the slightly lower computed skewness (0.596), as opposed to the measured skewness (0.789). A plot of these two distributions could be used to solve the inverse problem of determining the probability of the 30,000-cfs flood. It will be illustrated in Example 3-16.

The Log Pearson Type 3 Distribution

Analogous to the normal-lognormal distributions, when the three-parameter gamma distribution is applied to the logs of the random variables, it is customarily called the log Pearson type 3 (LP3) distribution. It plays an important role in hydrology because it has been recommended for application to flood flows by the U.S. IACWD (1982) in the committee's Bulletin 17B method. (Potential improvements to this method are discussed by Steidinger and Griffis [2008].) The shape of the LP3 is quite flexible due to its three parameters; an example for the Siletz River is shown in Figure 3-15.

Its use is entirely analogous to the lognormal discussed earlier; however, the moments of the transformed and untransformed variables will not be related here. Instead, the data are transformed by taking logarithms (either base 10 or natural), and the gamma-3 distribution is applied exactly as in the preceding section. This means that the magnitudes can readily be computed for return periods shown in Table 3-4, but the inverse problem of determining the return period (or CDF) corresponding to a given magnitude should be done graphically or with the tables mentioned in the preceding section or through the use of spreadsheet functions for the gamma distribution.

APPLICATION OF LP3 DISTRIBUTION TO FLOOD FLOWS**EXAMPLE 3-13**

Statistics of the logs of the Siletz River are given in Example 3-3; the weighted skewness (to be used strictly for flood frequency analysis) is -0.124 . For the 100-yr flood, linear interpolation in Table 3-4 gives $K = 2.234$. Thus, if $y = \log_{10} Q$,

$$y_{100} = 4.2921 + 2.234 \cdot 0.12905 = 4.5804$$

and the 100-yr flow is

$$Q_{100} = 10^{4.5804} = 38,060 \text{ cfs.}$$

Magnitudes predicted by the LP3 for this example will be graphed in Example 3–16, and the inverse problem (e.g., determining the return period of a flood of 30,000 cfs) will be considered at that time.

Extreme Value Distributions

Several additional distributions for hydrologic variables could be considered, and the most important of which, perhaps, are the **extreme value** (EV) family of distributions (Gumbel, 1958). These include the EV I, or Gumbel distribution, the EV II, the EV III (also known as the Weibull distribution when used for minima, such as droughts), and the generalized extreme-value distribution (GEV), which incorporates the three types just listed as special cases (Jenkinson, 1955; Stedinger et al., 1993). Although these distributions are often used in hydrology, space does not permit further examination in this text; see the references just cited as well as Benjamin and Cornell (1970) and Kite (1977).

One relationship sometimes encountered in hydrology, however, is the concept of the return period of the mean annual flood. The return period of the mean could be evaluated using any of the distributions discussed in this chapter, by $T(\mu) = 1/[1 - F(\mu)]$. Using the Gumbel (EV I) distribution, it is found that

$$F(\mu) = 0.57 \quad (3-69)$$

and

$$T(\mu) = \frac{1}{1 - F} = 2.33 \text{ yr,} \quad (3-70)$$

a value often encountered in the hydrologic literature. If only a plot of flow vs. return period is available, this relationship is sometimes used as a crude estimate of the mean of the flows, assuming that they obey a Gumbel distribution.

EXAMPLE 3-14

COMPARISON OF 100-YR FLOOD ESTIMATES

The 100-yr floods predicted by four distributions for flood frequency analysis discussed previously are compared in Table 3–5. For this relatively long record (75 yrs) and low measured skewness, the values are not far apart; in fact, all but the normal are likely to fall within the confidence intervals for each distribution (Stedinger et al., 1993). But there are differences, and so, which estimate should be used? A preliminary screening

Table 3–5 Comparison of Estimated 100-yr Floods and Measured and Predicted Skewness Values

Distribution	Predicted 100-yr Flood (cfs)	Predicted Skewness	Measured Skewness
Normal	34,620	0	0.789
Lognormal	39,110	2.86 [Eq. (3–60)] 0*	0.789 –0.156*
Gamma (Pearson 3)	38,010	0.789	0.789
Log Pearson 3	38,060	–0.124**	–0.124**

*Skewness of logarithms of the data.

**Weighted skewness of the logs.

can be performed on the basis of skewness. The untransformed data and their logarithms (Example 3–3) both exhibit nonzero skewness, so the normal distribution is suspect. All the other distributions have nonzero skewness, but only the gamma and log-gamma (LP3) theoretically match the data exactly (since they are three-parameter distributions). Finally, the LP3 is recommended by the IACWD. The best fit may also be evaluated on the basis of a comparison of the fitted (theoretical) CDF to the empirical CDF. This is done graphically, as explained in the next section.

Introduction

As in much of statistics, a visual inspection of the fit of the frequency distribution is probably the best aid in determining how well an individual distribution fits a set of data or which distribution fits “best.” Two questions must be addressed to plot the data: (1) What kind of graph paper (i.e., type of scaling) should be used? and (2) How should the data points be plotted (the question of **plotting position**)?

Probability Paper

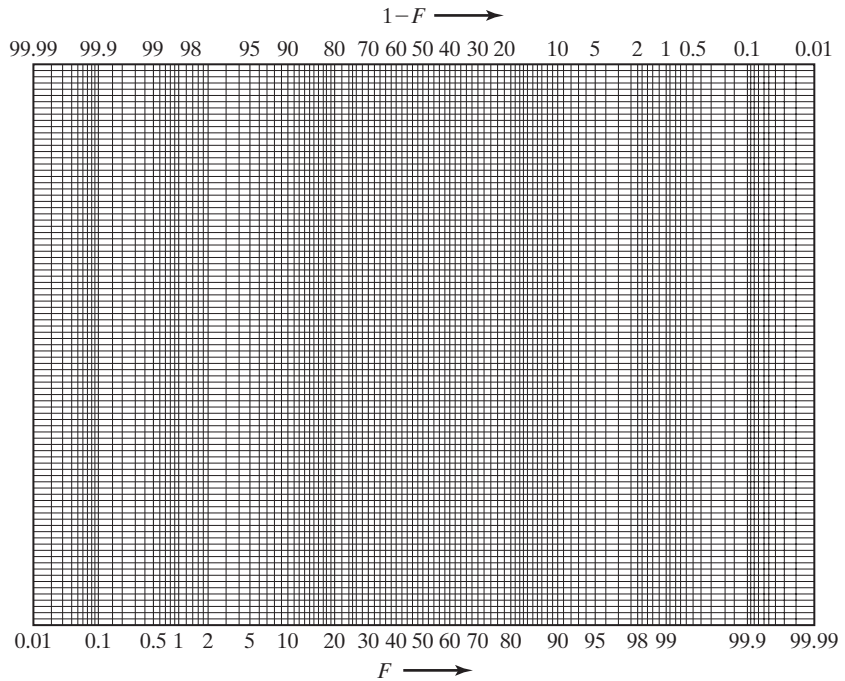
Although ordinary graph paper could be used to plot the CDF vs. the magnitude of the random variable, it is customary to try to use paper scaled such that the theoretical fit is a straight line. That is, it is desirable to plot magnitude x vs. some function of the CDF $h[F(x)]$ such that x vs. h is linear. If this can be done, one ordinate of the paper will be the magnitude and the other the function h , but this axis will be labeled with values of F , not h . Among other advantages, this permits a graphical fit of the distribution to the data simply by drawing a straight line through the plotted data points.

The frequency factor K [Eq. (3–43)] serves as the required function $h(F)$ as long as it is not also a function of skewness. For example, for the

3.6 GRAPHICAL PRESENTATION OF DATA

Figure 3-16

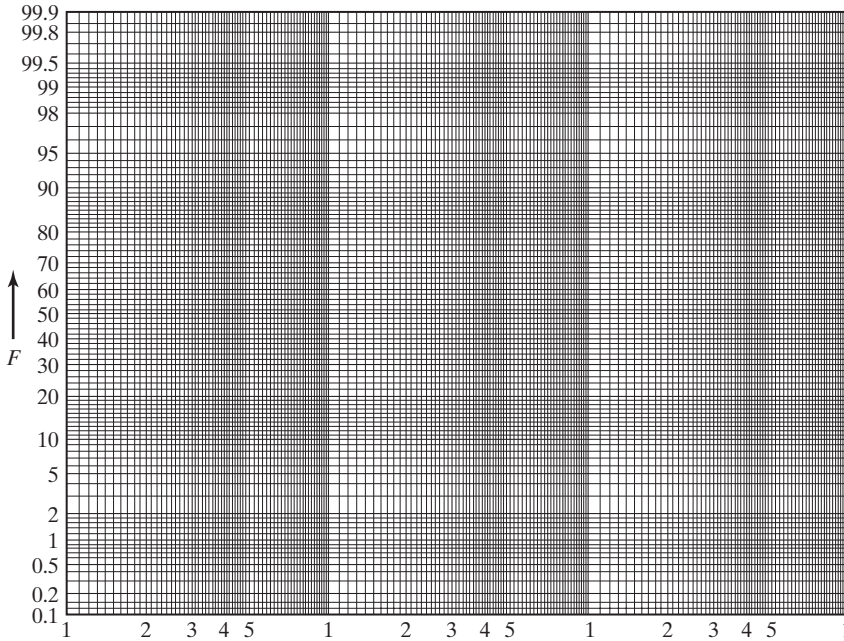
Normal probability paper.



normal distribution, $K = z$, the standard normal variate. Clearly, a plot of x vs. z is linear [Eq. (3-58)], and equal increments of z on one axis of probability paper can be labeled with the corresponding value of F for convenience. This is exactly how normal probability paper is constructed (Fig. 3-16). Log-normal paper is easily constructed also, either by plotting the logs on an arithmetic scale or, more commonly, by providing a log scale instead of an arithmetic scale for the magnitude of the variable (Fig. 3-17). A good source for normal, lognormal, and many other kinds of graph paper is Craver (1996). Excel does not provide probability scales of the type shown in Figures 3-16 and 3-17. One option for use with this spreadsheet is simply to plot magnitude (or log-magnitude) vs. z (found using the NORMINV function), to test the linearity of a fit. With more effort, normal probability paper may be constructed by the user by “drawing” the vertical lines shown in Figure 3-16 after determining their location from a computation of the corresponding z -value. The vertical lines are then labeled with their F -value. Finally, data are plotted on the graph against the hidden z -scale. This method was used to construct Figures 3-18 and 3-19 in this chapter. Finally, magnitude is sometimes plotted against $\log(T)$ or $\log(1-F)$ as an expedient, in the absence of probability paper. Of course, there will be no straight-line fit in this case (but see comment about the exponential distribution, below).

For the normal distribution, the straight fitted line passes through the estimate of the mean of the data at the 50% probability ($z = 0$) and through

Figure 3-17
Lognormal
probability paper.



the location $\bar{x} \pm S_x$ at the 84.1% or 15.9% frequencies [values of $F(x)$], respectively, from which $\bar{x}$ and S_x can be found graphically.

The problem is not quite as simple for the lognormal, since it is the logs, not the untransformed variables, which are distributed linearly vs. z . Thus, the 50% CDF value corresponds to the median of x , not the mean. To estimate the mean and standard deviation of the untransformed data from a graphical lognormal fit, the following procedure must be used.

1. Estimate the mean of the logs and the standard deviation of the logs using the procedure just described for the normal distribution. That is, the logarithm of the value plotted at 50% is the mean of the logs:

$$\bar{y} = \ln x_m = \ln x_{50} \quad (3-71)$$

and

$$S_y = \ln x_{84.1} - \ln x_{50} = \ln(x_{84.1}/x_{50}), \quad (3-72)$$

where

$$y = \ln x.$$

2. Knowing these moments of y , the moments of the untransformed variable x must be calculated from the relationships given in Equations (3-61) and (3-62).

This procedure will be illustrated later in Example 3-16.

Probability paper also exists for the extreme value distributions (Problem 3.20) but is not discussed herein. Unfortunately, no general probability paper can be constructed for the gamma or log-gamma distributions, since the frequency factor is a function of the skewness. Thus, a probability scale must be constructed for every value of C_s , which is feasible using computer programs for that purpose (Bobée and Ashkar, 1991). However, it is common to plot gamma or LP3 distributions on lognormal paper, on which they appear as smooth curves, with higher curvature for higher skewness. This will be illustrated in Example 3-16.

Finally, it may be mentioned that the exponential distribution has a very simple probability paper. If the exceedance frequency is $G(x) = 1 - F(x)$, then

$$G(x) = e^{-\lambda x} \quad (3-73)$$

and

$$\ln G = -\lambda x. \quad (3-74)$$

Thus, ordinary semilog paper serves as probability paper for the exponential distribution with G plotted on the log scale.

Plotting Position

To plot the flood (or other hydrologic) data, the data values must first be ranked from 1 to n (the number of years of record) in order of decreasing magnitude. Thus, magnitude $m = 1$ is the largest value and $m = n$ the smallest. The rank m and number of years of record n are then used to compute a plotting position, or empirical estimate of frequency F , or return period T .

In hydrology, the most common plotting position is the Weibull formula:

$$T = \frac{n + 1}{m} \quad (3-75)$$

and

$$F = 1 - \frac{m}{(n + 1)}, \quad (3-76)$$

which is analyzed in detail by Gumbel (1958). For example, the largest value from a 25-yr record would plot at a return period of 26 yr or a CDF of $25/26 = 0.962$.

However, the venerable Weibull formula has been criticized because it does not provide an estimate of the CDF F such that $E(F)$ equals the theoretical value for the m th largest out of n total samples for any underlying distribution other than the uniform, thus excluding all of the distributions commonly employed for flood frequency and other hydrologic analysis

(Cunnane, 1978). Instead, a generalized form first proposed by Gringorten (1963) may be used:

$$T = \frac{n + 1 - 2a}{m - a} \quad (3-77)$$

and

$$F = 1 - \frac{m - a}{n + 1 - 2a}. \quad (3-78)$$

The parameter a depends on the distribution and equals 0.375 for the normal (or lognormal) and 0.44 for the Gumbel, with a value of 0.40 suggested as a good compromise for the customary situation in which the exact distribution is unknown. Although the Weibull formula still finds much acceptance in hydrology, Equations (3-77) and (3-78) are better from a theoretical standpoint and will be used in the examples that follow, with parameter $a = 0.4$. For example, the largest value in a 25-yr record would plot with a return period of 42 yr and CDF = 0.976. Hirsch and Stedinger (1987) offer alternative plotting-position formulas for instances in which a continuous historical flood record is augmented by estimates of historical flood peaks prior to the period of continuous stream gaging. (Plotting-position computations are illustrated in Example 3-16.)

Confidence Limits, Outliers, and Zeros

Confidence limits are control curves plotted on either side of the fitted CDF, with the property that, if the data belong to the fitted distribution, a known percentage of the data points should fall between the two curves. Unfortunately, the computation of these limits differs for the different distributions; Kite (1977) and Stedinger et al. (1993) summarize the procedures to be used for each. Stedinger et al. also summarize and provide references for alternative procedures such as the probability plot correlation coefficient.

However, to illustrate the large uncertainty in frequency estimates, an approximate procedure used by Benjamin and Cornell (1970) will be demonstrated. They use the Kolmogorov–Smirnov (KS) goodness of fit statistic to plot confidence limits for the Gumbel and lognormal distributions, with the implication that it will serve as an approximate procedure for other distributions as well. Let $F_p(x)$ be the *predicted* value of the CDF. Then a confidence interval on the CDF can be constructed such that

$$\text{Prob}(F \leq F_u) = \text{Prob}(F \leq F_p + KS) = 1 - \alpha \quad (3-79)$$

and

$$\text{Prob}(F_l \leq F) = \text{Prob}(F_p - KS \leq F) = 1 - \alpha, \quad (3-80)$$

where KS is the Kolmogorov–Smirnov statistic at confidence level α , and subscripts u and l mean upper and lower, respectively. Values of KS are

Table 3-6 Kolmogorov-Smirnov Statistics

Sample Size	KS Value (fraction) for Confidence Level		
	$\alpha = 1\%$	$\alpha = 5\%$	$\alpha = 10\%$
5	0.67	0.56	0.51
10	0.49	0.41	0.37
15	0.40	0.34	0.30
20	0.35	0.29	0.26
25	0.32	0.26	0.24
30	0.29	0.24	0.22
40	0.25	0.21	0.19
Large n	$1.63/\sqrt{n}$	$1.36/\sqrt{n}$	$1.22/\sqrt{n}$

listed in Table 3-6 as a function of α and the sample size n . The $(1 - 2\alpha)$ percent confidence limits may be formed on $F(x)$ by

$$\text{Prob}(F_p - KS \leq F \leq F_p + KS) = 1 - 2\alpha. \quad (3-81)$$

For example, for a sample size of 40, 90% confidence limits may be placed on the predicted CDF by

$$\text{Prob}(F_p - 0.21 \leq F \leq F_p + 0.21) = 90.$$

These limits may be plotted on the graph of magnitude vs. CDF and control curves smoothed in for intermediate values; the curves can then be used to estimate confidence limits on magnitudes as well as CDF values. The procedure will be illustrated in Example 3-15.

The KS statistic is weak inasmuch as it is independent of the actual distribution being plotted. It is also a constant and does not reflect the additional uncertainty in predicted values of F at the extremes of the plotted points (i.e., $m = 1$). Finally, it cannot be used to compute F_u when $F_p + KS > 1.0$ or to compute F_l when $F_p - KS < 0$. The best procedure is to use the method appropriate to each distribution. The details are beyond the scope of this text; see Kite (1977) and Stedinger et al. (1993).

Outliers are data points that “depart significantly from the trend of the remaining data” (Interagency Advisory Committee on Water Data, 1982) and pose another problem in frequency analysis. Should they be included in the analysis or are they anomalies? Statistical tests are available (e.g., Interagency Advisory Committee on Water Data, 1982; see Problem 3.23), but in the end the decision about whether or not to retain an outlier is usually made subjectively on the basis of confidence in the individual data value.

The presence of zeros among the data is usually treated by means of a mixed distribution (Fig. 3-8). If there are n_0 zeros among n samples, the discrete probability $P(0) = \text{Prob}(Q = 0)$ is estimated as n_0/n . A continuous PDF is then fit to the remainder of the data using the moments of the non-zero data, but scaled such that its total mass is $1 - P(0)$ instead of 1.0. This

is done by simply multiplying values of the CDF by $1 - P(0)$. The base value of the CDF on a plot is thus $P(0)$. An example of this procedure is given by Haan (2002).

GRAPHICAL FIT OF LOGNORMAL DISTRIBUTION

EXAMPLE 3-15

The Siletz River data from Example 3-3 are to be used to fit the lognormal distribution and to plot the results. The fit has already been performed in Example 3-11 using the method of moments. Hence, the fitted distribution will be plotted using those results. Both Table 3-7 and Figure 3-18 were prepared using the Excel spreadsheet.

To plot the data points, the data of Table 3-2 are first ranked, as shown in Table 3-7. (Only every fifth point is tabulated in the middle of the ranked series, to save space.)

Return periods and CDF values are then assigned using Equations (3-77) and (3-78), also indicated in Table 3-7. To plot 90% confidence intervals, the theoretical (fitted) CDF is computed for each flow, and approximate values for F_u and F_l are computed using $KS = 1.36/\sqrt{n} = 0.157$ for $\alpha = 5\%$ and $n = 75$ yr (Table 3-6). Clearly, F_u cannot be computed for

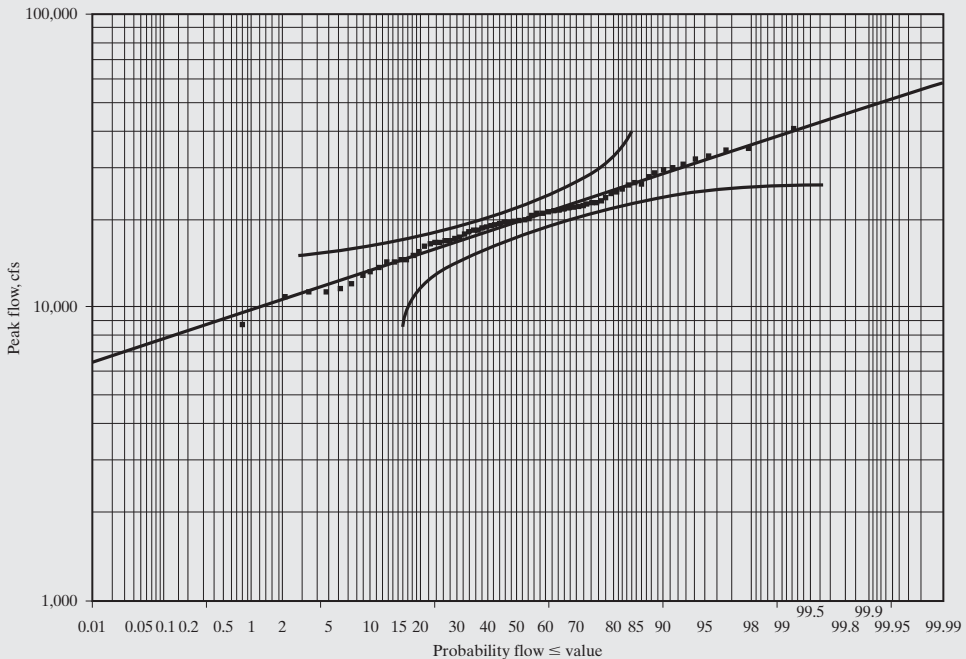


Figure 3-18

Lognormal plot of Siletz River flows, with 90% confidence intervals. Only every fifth value is plotted in the middle of the ranked series, for additional clarity.

Table 3-7 Tabulated Siletz River Data for Lognormal Plot

Water Year	Rank (m)	Empirical Return Period	Empirical CDF	Peak Flow (cfs)	Fitted CDF	Upper Confidence Limit*	Lower Confidence Limit**
1999	1	125.3	0.9920	40,500	0.9927		0.836
1996	2	47.0	0.9787	34,700	0.9728		0.816
1931	3	28.9	0.9654	34,100	0.9689		0.812
1965	4	20.9	0.9521	32,200	0.9527		0.796
1972	5	16.3	0.9388	31,800	0.9484		0.791
1928	6	13.4	0.9255	30,700	0.9347		0.778
1938	7	11.4	0.9122	30,100	0.9258		0.769
1949	8	9.9	0.8989	29,000	0.907		0.749
1934	9	8.7	0.8856	28,700	0.901		0.744
1947	10	7.8	0.8723	28,000	0.885		0.728
1943	11	7.1	0.8590	26,500	0.845		0.688
1981	12	6.5	0.8457	26,500	0.845		0.688
1963	13	6.0	0.8324	26,300	0.839	0.996	0.682
1942	14	5.5	0.8191	25,400	0.809	0.966	0.652
1953	15	5.2	0.8059	24,600	0.778	0.935	0.621
1997	20	3.8	0.7394	22,700	0.690	0.847	0.533
1932	25	3.1	0.6729	21,800	0.640	0.797	0.483
1955	30	2.5	0.6064	21,200	0.605	0.762	0.448
1987	35	2.2	0.5399	20,000	0.528	0.685	0.371
1927	40	1.9	0.4734	19,500	0.494	0.651	0.337
1925	45	1.7	0.4069	18,800	0.445	0.602	0.288
1971	50	1.5	0.3404	18,100	0.395	0.552	0.238
1926	55	1.4	0.2739	16,800	0.302	0.459	0.145
1937	60	1.3	0.2074	16,100	0.254	0.411	0.0974
1989	61	1.2	0.1941	15,400	0.209	0.366	0.0518
1935	62	1.2	0.1809	15,000	0.184	0.341	0.0273
1969	63	1.2	0.1676	14,500	0.156	0.313	
1980	64	1.2	0.1543	14,500	0.156	0.313	
1959	65	1.2	0.1410	14,200	0.139	0.296	
1960	66	1.15	0.1277	14,200	0.139	0.296	
1985	67	1.13	0.1144	13,600	0.110	0.267	
1941	68	1.11	0.1011	13,200	0.0919	0.249	
1944	69	1.10	0.0878	12,800	0.0760	0.233	
1993	70	1.08	0.0745	12,000	0.0495	0.207	
1930	71	1.07	0.0612	11,500	0.0365	0.194	
1984	72	1.05	0.0479	11,300	0.0320	0.189	
1929	73	1.04	0.0346	11,200	0.0299	0.187	
1992	74	1.02	0.0213	10,800	0.0225	0.180	
1977	75	1.01	0.0080	8630	0.0029	0.160	

*Fitted CDF + KS value of 0.157.

**Fitted CDF - KS value of 0.157.

$F \geq 0.84$ (i.e., for $F \geq 1 - KS$) and F_I cannot be computed for $F \leq 0.16$. Finally, the data, fitted distribution, and confidence limits are plotted in Figure 3-18.

The lognormal is an excellent fit and could be fit by eye by drawing a straight line through the data points. However, here the fit of Example 3-11 is shown, for which only two points need be computed to draw the straight line: The 50% magnitude (the median) is

$$Q_m = Q_{50} = 10^{\bar{y}} = 10^{4.2921} = 19,590 \text{ cfs} \quad (y = \log_{10} Q).$$

A second point for the straight line is found by adding the standard deviation of the logs to the mean of the logs:

$$y_{84.1} = \bar{y} + S_y = 4.2921 + 0.12905 = 4.4216.$$

Thus,

$$Q_{84.1} = 10^{4.4216} = 26,400 \text{ cfs}.$$

Two points are now available with which to draw the straight fitted CDF. The slightly poorer fit of the distribution at the low end is not of concern when the analysis is for flood conditions. In fact, one improvement to standard flood frequency protocols that has been suggested by the National Research Council (1988) is to focus only on the extremes (upper tail), as opposed to, or even to the exclusion of, central characteristics, in the common situation in which the overall fit of a distribution is not as good as shown in Figure 3-18.

What are the mean and standard deviation of the untransformed flows using the graphical fit? This question may make more sense when the fit has been performed by eye instead of analytically as in Example 3-11. Nonetheless, assume that the mean and standard deviations of the base 10 logs of the flows are found to be 4.292 and 0.129, respectively, by an inverse of the procedure explained in the previous paragraph. Then $\bar{Q}$ and S_Q must be found using the relationships of Equations (3-61) and (3-62). The log statistics are first converted to natural logs (if necessary, as it is in this example):

$$\ln \bar{Q} = 2.3026 \cdot 4.292 = 9.878$$

and

$$S_{\ln Q} = 2.3026 \cdot 0.129 = 0.297.$$

Equations (3-61) and (3-62) then give

$$\bar{Q} = e^{9.878 + 0.297^2/2} = 20,380 \text{ cfs}$$

and

$$S_Q = 20,380(e^{0.297^2} - 1)^{0.5} = 6190 \text{ cfs}$$

These values are close to 20,450 and 6090 cfs, respectively, for the mean and standard deviation computed from the data (Example 3–3), but not the same. Once again, the log of the mean is not the mean of the logs!

The 90% Kolmogorov–Smirnov confidence limits form very wide bands about the fitted distribution. On the one hand, this indicates that the fit is acceptable (at the 90% level), since all of the plotted points fall within the bands. On the other hand, the bands reflect the extreme uncertainty of the probability estimates. For instance, considering a flow of 25,000 cfs, from Figure 3–18 the probability is 90% that the true CDF for this flow lies between 62% and 95%. Put another way, the probability is 90% that the true return period for a flow of 25,000 cfs lies between 2.6 and 20 yr—a very wide interval!

Comparison of Fits

The question of which distribution gives the best fit may be addressed quantitatively using measures such as the chi-square statistic and Kolmogorov–Smirnov test (Benjamin and Cornell, 1970; Haan, 2002), but these tests are seldom helpful in discriminating among different distributions, because their confidence limits are so large that both tend to lead to acceptance of the hypothesis that the distribution fits the data. See, for example, the very wide 90% confidence intervals plotted in Figure 3–18 using the *KS* statistic. An alternative is to use heuristic measures of goodness of fit, such as the average of absolute values of deviations between the fitted and plotted CDF (Benson, 1968). Another possibility is to include the third and fourth moments of the data in the analysis, since these may be used to categorize different distributions (Harr, 1977). But the problems of estimation of higher moments have already been discussed. Better alternatives (in the sense of greater discrimination among alternative distributions) are the probability plot correlation coefficient method and L-moment tests, summarized by Stedinger et al. (1993). In the end, the decision is often subjective and based on a preference for the underlying mechanism of one distribution vs. another. That is, the engineer may prefer the multiplicative mechanism of the lognormal distribution or the empiricism of the gamma or log Pearson type 3 distributions.

Graphical comparison of the fitted and plotted CDFs is of considerable use in the decision. King (1971) has shown the form that several CDFs exhibit when plotted on the probability paper of another distribution, and Reich and Renard (1981) apply these shapes to hydrology. For example, if the data are plotted on lognormal paper and a Gumbel distribution fits the data, the data should appear with curvature. Unfortunately, the gamma and log Pearson 3 distributions cannot be assessed in this manner, since they have no probability paper.

Instead of trying several distributions and attempting to select the best fit, one could simply accept the recommendations of the U.S. IACWD (1982) (formerly the U.S. Water Resources Council; Benson,

1968) that the LP3 distribution is best suited for flood frequency analysis. On the other hand, the log-Gumbel (extreme-value type II) was determined by the Natural Environment Research Council (1975) to be preferable for flood frequency analysis in Great Britain. Individual rivers are likely to vary in their optimal distribution, and the question of which distribution to use for floods and myriad other hydrologic variables will always remain open.

GRAPHICAL COMPARISON OF FOUR DISTRIBUTIONS

EXAMPLE 3-16

Which distribution is best for the data for the Siletz River? One very qualitative assessment can be made from the four PDFs shown in Figure 3-15. From the figure, we see little difference among them. However, common practice is to compare the CDFs on probability paper; and for this purpose, all the distributions may be plotted together on lognormal paper for another qualitative assessment. Several predicted magnitudes are needed, since all but the lognormal will plot as smooth curves. These calculations are shown in Table 3-8 (including more than just the required two points for the lognormal). The resulting plots are shown in Figure 3-19 along with selected data points (only every fifth data point is plotted in the middle of the distribution, in order to have a better view of the four fitted distributions).

Table 3-8 Computation of Estimated Flood Magnitudes, Siletz River, Near Siletz, Oregon, Water Years 1925-1999

Return Period	CDF	Normal		Lognormal		Gamma-3		Log Pearson 3	
		$K = z$	Q (cfs)	$K = z$	Q (cfs)	K	Q (cfs)	K	Q (cfs)
500	0.998	2.878	37,980	2.878	46,080	3.837	43,810	2.728	44,070
200	0.995	2.576	36,140	2.576	42,120	3.303	40,560	2.459	40,690
100	0.99	2.326	34,620	2.326	39,110	2.884	38,010	2.235	38,060
50	0.98	2.054	32,960	2.054	36,070	2.448	35,360	1.987	35,360
25	0.96	1.751	31,110	1.751	32,960	1.990	32,570	1.707	32,540
10	0.9	1.282	28,260	1.282	28,670	1.336	28,590	1.268	28,550
5	0.8	0.842	25,580	0.842	25,160	0.781	25,210	0.847	25,200
2	0.5	0	20,450	0	19,590	-0.130	19,660	0.021	19,710
1.0101	0.009999	-2.326	6290	-2.326	9810	-1.741	9850	-2.417	9550

This table was prepared using Excel. Frequency factors (K) for the normal distribution were obtained using Excel function NORMSINV and so might differ slightly from linear interpolation in Table D-1. Frequency factors for the gamma and LP3 distributions were obtained using Excel function GAMMAINV and so might differ slightly from linear interpolation in Table 3-4. All estimated flows are rounded to the nearest 10 cfs.

Mean of flows = 20,452 cfs

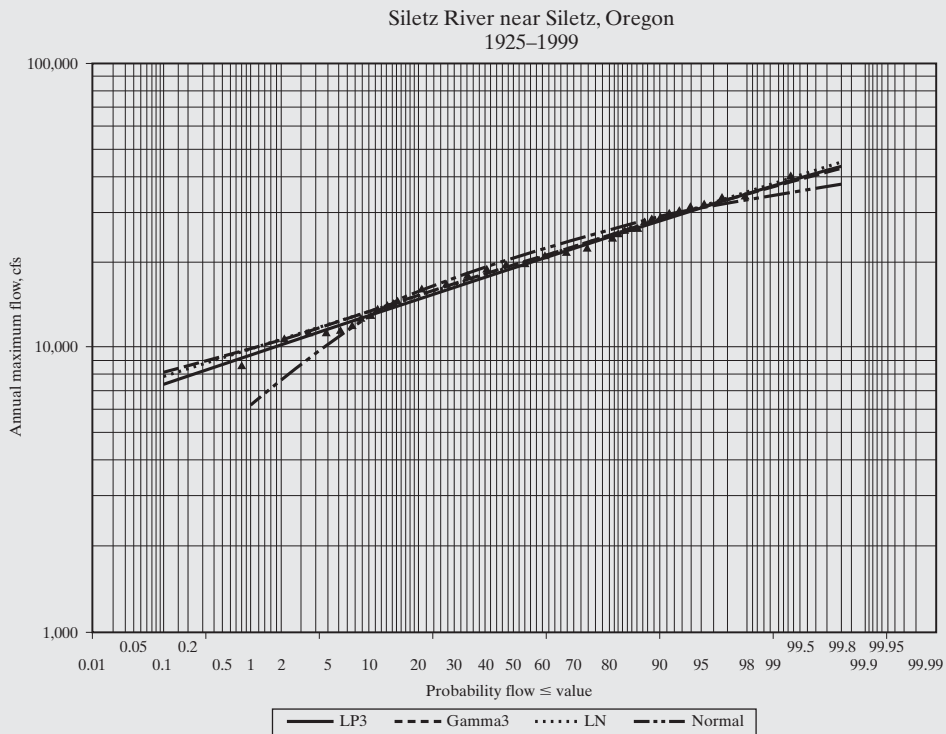
Standard deviation of flows = 6089 cfs

Skewness of flows = 0.789

Mean of $\log_{10}$ (flows) = 4.2921

Standard deviation of $\log_{10}$ (flows) = 0.12905

Weighted skewness [Eq. (3-41)] of $\log_{10}$ (flows) = -0.1242.

**Figure 3–19**

Comparison of four fitted CDFs for Siletz River flows, 1925–1999.

All but the normal distribution are good fits for the Siletz River. There is no noticeable curvature to the data points; in fact, the lognormal fit is clearly very good. The choice would likely boil down to an individual preference for one mechanism or recommendation over another, as discussed above. Clearly, the LP3 distribution, as recommended in Bulletin 17B, works well for the Siletz. However, of the three good fits, the lognormal is easiest to apply and might be best known to many engineers.

It is important to realize that, were confidence limits shown (as on Fig. 3–18), they would be very wide at the upper limit of the CDF, reflecting the uncertainty of predicting the magnitude of, say, the 100-yr flood from only 75 yr of data. The uncertainty can be quantified by computing the standard deviation of the estimates at the extremes, using a different procedure for each distribution (Kite, 1977; Stedinger et al., 1993). One way to reduce the uncertainty is to use a longer period of record (not possible) or a regional analysis, in which regional streamflow data are

aggregated and used in a regression-type analysis (Section 3.7). The National Research Council (1988) study describes several options for improvements of frequency estimates, including regional analysis, better structure in models, and focusing on the upper extremes (for floods).

Finally, plots such as Figure 3–19 can be used to interpolate for flood frequencies for the gamma and LP3 distributions, as suggested in Examples 3–12 and 3–13, in lieu of the use of tables or spreadsheet functions. Given the uncertainty inherent in flood frequency estimates, two-significant-figure accuracy from a plot is generally accurate enough. For instance, the nonexceedance probability of a flow of 30,000 cfs is approximately 93% by any of the four distributions, with a corresponding return period of about 14 years.

Computer Programs

All the computations, tables, and figures in this chapter were prepared easily in Excel. Functions for obtaining the cumulative normal distribution and its inverse and the cumulative gamma distribution and its inverse are particularly helpful. On the other hand, preparation of probability paper plots of the type of Figures 3–18 and 3–19 takes more effort, since the vertical lines of the probability scale must be inserted (“drawn”) and labeled by the user. Nonetheless, the task is straightforward.

Flood frequency software is available from various sources. The Hydrological Frequency Analysis (HFA) program may be purchased as an adjunct to the text by Bobée and Ashkar (1991) for analysis and probability plots of various formulations of the gamma and LP3 distributions. The Bulletin 17B procedure of the IACWD (1982) has been programmed by the Army Corps of Engineers, Hydrologic Engineering Center (HEC), and is available as the program HEC-SSP (HEC Statistical Software Package). See the HEC website (<http://www.hec.usace.army.mil/>) for information about how to obtain the software. A similar program for Bulletin 17B analysis, PEAKFQ, has been prepared by the U.S. Geological Survey and may be downloaded from its website, http://water.usgs.gov/software/lists/surface_water. At the same location, the program SWSTAT may be downloaded, which performs many different kinds of statistical and related analyses on time-series data, including preparation of flow-duration curves, trend analysis, and a simplified form of the Bulletin 17B analysis.

Flood frequency estimates illustrated in this chapter rely upon a record of flows measured at one station. The shorter the record, the more uncertainty in the frequency estimates. See, for example, how the *KS* values of Table 3–6 increase with decreasing *n*. **Regional analysis** is one way in which problems of short records are addressed. A summary of literature is provided by Stedinger et al. (1993), and advanced techniques are provided by

3.7 REGIONAL ANALYSIS

Hosking and Wallis (1997). One alternative is to perform regional estimates of key statistics, such as skewness, illustrated with the discussion of Figure 3–10. Another alternative is a regional regression: T -year floods are computed for as many streams as possible using the LP3 distribution, sometimes with the aid of simulation models, and their magnitudes regressed against catchment characteristics. The relationship often takes the form of a multiple regression analysis (power-function form), in which catchment area is usually the most important independent variable. In this way, uncertainty in a short-record-length estimate at one station is balanced against a longer-record estimate at other stations. Such relationships are also good ways in which to estimate flood peaks at ungaged locations. The U.S. Geological Survey has prepared these relationships for many locations in the United States. Examples include Harris et al. (1979) and Cooper (2005) for western Oregon, Sauer et al. (1983) for urban areas in the United States, Laenen (1983) for urban areas in western Oregon, and Franklin and Losey (1984) for Tallahassee, Florida. An automated program, NFF (for National Flood Frequency), which incorporates all published results for the United States, has been published by the USGS (Ries and Crouse, 2002). Local offices of the USGS should be contacted for access to studies for specific regions.

The nationwide urban equations of Sauer et al. (1983) depend on local estimates of rural peaks and cannot be demonstrated here. However, as an example, the USGS (Franklin and Losey, 1984) has performed a flood frequency analysis on 15 basins in the Tallahassee, Florida, area using a combination of continuous simulation and regression. The resulting regression equation for most of the Tallahassee area is

$$Q_p(T) = C_p A^a I A^b, \quad (3-82)$$

where

$$\begin{aligned} Q_p &= \text{predicted flood peak for return period } T \text{ (cfs),} \\ A &= \text{drainage area (mi}^2\text{),} \\ IA &= \text{impervious area as percentage of total area } A, \end{aligned}$$

and C_p , a , and b are coefficients that are functions of return period, as given in Table 3–9. The 15 basins ranged from 0.2 to 16 mi² and imperviousness ranged from 5.8% to 54%. Coefficients of determination (R^2) for the multiple regression ranged from 0.97 to 0.99 for the indicated equations, but standard errors of estimate were 18% to 30%. That is, there is a 68% chance (one standard deviation on either side of the mean, for the normal distribution) that the unknown “true” flood estimate will fall within 18% to 30% of the true mean. The regression predictions are mainly for use in ungaged catchments. For example, for the 3.44 mi² McGinnis Arm Catchment in Tallahassee, the imperviousness percentage is 28.3, for which Equation (3–82) yields a 10-yr peak flow of 1850 cfs.

Table 3-9 Coefficients for Flood Peak Prediction in Tallahassee, for Equation (3-82)

Return Period (yr)	C_p	a	b
2	10.7	0.766	1.07
5	24.5	0.770	0.943
10	39.1	0.776	0.867
25	63.2	0.787	0.791
50	88.0	0.797	0.736
100	118.0	0.808	0.687
500	218.0	0.834	0.589

Hydrologic analysis often wanders further into statistics and time-series analysis. It is assumed that the reader is familiar with common tools of statistics such as regression and correlation analysis, which are described in almost any statistics text. **Time-series analysis** deals with the treatment of data while retaining their temporal sequence, with possible serial correlation and other underlying relationships (e.g., periodic cycles) among the individual data points. Priestly (1981) and Chatfield (2003) describe techniques, and hydrologic applications may be found in texts such as Salas et al. (1980), Bras and Rodriguez-Iturbe (1985), and Salas (1993). An excellent reference for application of a broad range of statistical techniques to water resources problems is provided by Helsel and Hirsch (1992).

3.8 RELATED TOPICS

Modern techniques for analysis of water resources systems often rely upon long-term (continuous) simulations for optimization of management options. For example, many different operating policies may be feasible for a reservoir or for a series of hydraulic structures in a watershed. What policies are optimal for various goals? Simulation models are often used to test different design and management options and are driven either by historic time series of rainfall and/or flows or by **synthetic rain** and/or **flow sequences**. A synthetic time series is one generated statistically that maintains key statistics of a measured time series, typically the mean, variance, and serial correlation, although additional constraints can also be met. An unlimited number of data points may be generated synthetically, compared to the limited historical record, thus, the usefulness of the synthetic sequences for simulations. For example, a **Markov model** for generation of a series of *normally* distributed synthetic flows is:

$$Q_{n+1} = \bar{Q} + r_1(Q_n - \bar{Q}) + (1 - r_1^2)^{0.5} S_Q z, \quad (3-83)$$

where

$$\begin{aligned} Q_{n+1} &= \text{flow in time period } n + 1, \\ Q_n &= \text{flow in time period } n, \end{aligned}$$

$\bar{Q}$ = mean of the flows,

r_1 = lag-1 serial correlation coefficient between flows in time period $n + 1$ vs. flows in time period n (obtained simply by linear regression or through programs for time-series analysis and typically a small, positive fraction, e.g., 0.05–0.25),

r_1^2 = coefficient of determination of the serial correlation—i.e., the fraction of variance in Q_{n+1} accounted for by the correlation with Q_n ,

S_Q = unbiased standard deviation of the flows, and

z = normally distributed random variable with zero mean and unit variance—i.e., an $N(0,1)$ distribution.

Flow sequences generated by Equation (3–82) will preserve the mean, standard deviation, and lag-1 serial correlation coefficient of the flows. Random numbers may be generated for almost any distribution (Abramowitz and Stegun, 1964), and most spreadsheets provide functions to generate normally distributed random numbers. Note that logarithms of flow sequences could be generated similarly, thus simulating a lognormally distributed time series. In this case, however, statistics of the logs should be based on the moment relationships of Equations (3–61) and (3–62) (Fiering and Jackson, 1971), as in Example 3–11(c), not on the basis of the transformed data. Options for other frequency distributions are given in the cited references.

The first significant application of synthetic flow sequences was provided by Maass et al. (1962); more complicated time-series models are provided by Box and Jenkins (1976). Classical Markovian procedures are presented by Fiering and Jackson (1971) and Haan (2002). A summary of methods for use in hydrology is provided by Salas (1993).

SUMMARY This chapter contains only a sampling of possible topics in frequency analysis. Other topics of particular importance are improved estimation procedures by the method of maximum likelihood and L-moments (see Section 3.3), and confidence limits and uncertainty (see Section 3.6). The reader may refer to the cited references for additional information.

In this chapter, introductory definitions related to description of data and probability theory are followed by methods for fitting theoretical frequency distributions to measured data. One discrete and five continuous distributions are described and applied. The emphasis is on flood frequency analysis, but the procedures can be applied to any set of independent hydrologic data, such as rainfall, stage, annual flow volumes, temperatures, and water quality parameters. Apart from the recommended use of the log-Pearson 3 distribution for flood frequency analysis in

the United States (IACWD, 1982), there is no consensus regarding distributions most suited to any particular variable; however, for storm runoff quality, studies have shown that storm runoff event mean concentrations (i.e., flow-weighted average storm event concentrations) are almost universally distributed lognormally (Driscoll, 1986).

Although most of the distributions discussed can also be applied to minima (e.g., droughts), the parameter estimates may vary from those given for the maxima to accommodate the possible negative skewness of the droughts. For use of any distribution other than the normal for minima, reference should be made to Gumbel (1958), Benjamin and Cornell (1970), Stedinger et al. (1993), or Haan (2002) for parameter estimation.

PROBLEMS

- 3.1.** This chapter employs Siletz River data for the continuous period 1925–1999. As of this writing, peak flow data through water year 2010 are available on the USGS Oregon surface water data webpage <http://waterdata.usgs.gov/or/nwis/sw>, including for Gage Number 14305500, which is the Siletz River. You will be asked in different problems in this chapter to update the Chapter 3 examples by using the 86-yr record, 1925–2010.
- (a) Download the peak flow data for the Siletz River, and enter the data into a spreadsheet.
 - (b) Plot the 86 years of peak flows (1925–2010) and a 5-yr running mean vs. their water year, to update Figure 3–2. Comment in general about the appearance of this time series, in the manner of the discussion of Figure 3–2.
 - (c) Using the updated data, develop new relative frequency and cumulative frequency histograms; that is, update Figures 3–4 and 3–5.
- 3.2.**
- (a) Use the data found in Problem 3.1 to calculate the mean, standard deviation, and skew coefficient (Eqs. 3–37, 3–38, and 3–40) of the updated Siletz River data (1925–2010).
 - (b) Repeat part (a) using the logs (base 10) of the Siletz River data.
 - (c) Develop the weighted skewness of the logs according to the Bulletin 17B protocol. That is, redo Example 3–3. At the conclusion of this problem, you should have an updated version of Table 3–2.
- 3.3.** A temporary cofferdam is being designed to protect a 5-yr construction project from the 30-yr flood. What is the probability that the cofferdam will be overtopped:
- (a) at least once during the 5-yr project,
 - (b) not at all during the project,
 - (c) in the first year only,
 - (d) in the fourth year and fifth year exactly?

- 3.4.** A recreational park is built near Buffalo Creek. The stream channel can carry $200 \text{ m}^3/\text{s}$, which is the peak flow of the 5-yr storm of the watershed. Find the following.
- The probability that the park will flood next year
 - The probability that the park will flood at least once in the next 20 yr
 - The probability that the park will flood three times in the next 20 yr
 - The probability that the park will flood 10 times in the next 20 yr

Problems 3.5 through 3.8 refer to the updated 1925–2010 Siletz River data developed in Problems 3.1 and 3.2. For consistency, assume that the following moments are valid for the period water years 1925–2010:

	Original Data (cfs)	Log ₁₀ Data (log cfs)
Mean	20,796	4.29217
Standard Deviation	7386	0.1527
Station Skewness	1.341	−0.3510
Weighted Skewness		−0.2773

- 3.5.** Assume that the Siletz River data are normally distributed. Find the following.
- Peak flow of the 100-yr flood
 - Peak flow of the 50-yr flood
 - Probability that a flood will be less than or equal to 30,000 cfs
 - Return period of the 30,000-cfs flood
- 3.6.** Assume that the Siletz River data are lognormally distributed. Find the following.
- Peak flow of the 100-yr flood
 - Peak flow of the 50-yr flood
 - Probability that a flood will be less than or equal to 30,000 cfs
 - Return period of the 30,000-cfs flood
- 3.7.** Assume that the Siletz River data may be fit by a three-parameter Gamma (Pearson 3) distribution. Find the following.
- Peak flow of the 100-yr flood
 - Peak flow of the 50-year flood
- 3.8.** Assume that the Siletz River data may be fit by a log-Pearson 3 (LP3) distribution. Find the following.
- Peak flow of the 100-yr flood
 - Peak flow of the 50-yr flood
- 3.9.** Generate a new flood frequency plot for the updated Siletz River data, 1925–2010. That is, generate an updated version of Figure 3–19, but only plot the Gamma-3 and LP3 fits. You may omit some data points in the middle of the ordered series to ease crowding. If you are unable to obtain lognormal

probability paper, a plot of magnitude vs. $\log(T)$ or magnitude vs. $\log(1-F)$ can serve as a substitute. That is, plot magnitude on the arithmetic scale vs. T or $1-F$ on the log scale on semilog paper or in Excel.

- 3.10.** Using graphs from Problem 3.9 and additional computations as appropriate, estimate the return period and nonexceedance probability, $F(Q)$, of a flood of magnitude 30,000 cfs for the Siletz River, 1925–2010.
- 3.11.** Match the letters on the right with the numbers on the left to complete the mathematical statements about PDF properties. Assume that x is a normally distributed annual occurrence.

1. $\int_{\mu}^{\mu} f(x) dx = \square$	(a) Standard deviation
2. $\int_{\square}^{\infty} f(x) dx = 0.02$	(b) Median
3. $\int_{\mu}^{\mu+\square} f(x) dx = 0.34$	(c) 0
4. $\int_{-\infty}^{\square} f(x) dx = 0.5$	(d) $P(m_1 \leq x \leq m_2)$
5. $\int_{m_1}^{m_2} f(x) dx = \square$	(e) 50-yr magnitude
	(f) Variance
	(g) $F(x)$

- 3.12.** The gamma distribution may be written in many different (but mathematically equivalent) forms. Excel uses the following form for the two-parameter gamma distribution in its functions $\text{GAMMADIST}(x_1) \equiv F(x_1)$ and $\text{GAMMAINV}(F) \equiv x_1$:

$$F(x_1, \alpha, \beta) = \int_0^{x_1} \frac{1}{\beta^\alpha \Gamma(\alpha)} x^{\alpha-1} e^{-x/\beta} dx, \quad x \geq 0.$$

The three-parameter gamma distribution with lower bound x_0 is easily evaluated by replacing the lower and upper limits by x_0 and $x_1 - x_0$, respectively. Method-of-moments estimates for the three parameters are

$$\begin{aligned}\alpha &= 4/C_s^2 \\ \beta &= S_x \cdot C_s/2 = S_x/\sqrt{\alpha} \\ x_0 &= \bar{x} - \alpha\beta = \bar{x} - \frac{2S_x}{C_s}.\end{aligned}$$

If $C_s < 0$, β will be negative (not allowed in Excel). In this case, $|\beta|$ is used in the Excel functions, and GAMMADIST returns $1 - F$, and the argument of GAMMAINV is $1 - F$. Method-of-moments estimates for the two-parameter gamma distribution are:

$$\alpha = \bar{x}^2 / S_x^2 = 1 / CV^2$$

$$\beta = S_x^2 / \bar{x}$$

The skewness of the two-parameter gamma distribution is

$$C_s = 2 / \sqrt{\alpha} = 2CV,$$

which provides a way of computing frequency factors, as described below.

Of course, moments are computed using logarithms of the data for the LP3 distribution. For given values of α and β , Excel function GAMMADIST returns $F(x_1)$ given x_1 , and GAMMAINV returns x_1 given F . Letting $x_1 = \bar{x} + K \cdot S_x$, where $K(F, C_s)$ is the frequency factor, and using the parameter estimates given above, the upper limit of the integral can be manipulated to give

$$K(F, C_s) = G_{inv} \cdot C_s / 2 - 2 / C_s,$$

where $G_{inv} = \text{GAMMAINV}(F, \alpha, \beta)$. To evaluate K , set $\beta = 1$ and $\alpha = 4 / C_s^2$. Hence, frequency factors can be computed only as a function of the CDF and skewness for any values of either parameter, and Table 3-4 can be avoided. For negative skewness, the symmetry of the distribution is exploited, and the same relationship holds for K , but with $1 - F$ as the argument of GAMMAINV, instead of F . (The negative sign of C_s is retained in the equation.) For the special case of $C_s = 0$, $K = z =$ standard normal variate.

- (a) Compute frequency factors for $T = 2$ yr and 100 yr and for $C_s = +0.5$ and -0.5 . Compare the four values with the values given in Table 3-4.
- (b) Repeat Problems 3.7, 3.8, and 3.10 using Excel functions GAMMADIST and GAMMAINV.

3.13. The total annual runoff for a small watershed was determined to be approximately normal with a mean of 360 mm and a variance of 3000 mm². Determine the probability that the total runoff from the basin will exceed 250 mm in all four of the next consecutive four years.

3.14. Data for Cypress Creek for the period 1951–2010 are listed on the USGS website. Following the link below, create a table of yearly peak streamflows, and then rank the data. Using these data, develop a relative frequency histogram and a cumulative frequency histogram for Cypress Creek, 1951–2010. Use a class interval of 2000 cfs.

USGS National Water Information System for Gage # 0806900: <http://waterdata.usgs.gov/tx/nwis/dv>

- 3.15.** (a) Use the data found in Problem 3.14 to calculate the mean, standard deviation, and skew coefficient (Equations 3-37, 3-38, and 3-40) of the Cypress Creek data (1951–2010).
- (b) Repeat part (a) using the log (base 10) of the Cypress Creek data.

Problems 3.16 and 3.17 refer to the Cypress Creek data found in Problem 3.14

- 3.16.** Assume that the Cypress Creek data for the period 1951–2010 are normally distributed. Find the following.
- (a) Peak flow of the 25-yr flood
 - (b) Peak flow of the 5-yr flood
 - (c) Probability that a flood will be less than or equal to 2000 cfs
 - (d) Return period of the 2000 cfs flood
- 3.17.** Assume that the Cypress Creek data for 1981–2010 fit a log Pearson 3 distribution (statistics of base 10 logs are $C_s = -0.788$, mean = 3.8262, Var = 0.05104, and st. dev. = 0.2259). Find the peak flow of the 100-yr flood, and compare it with the peak flow using the entire dataset (14,757 cfs peak 100-yr flood using entire dataset). Explain the difference, knowing that 95% of residential development along Cypress Creek occurred after 1965.
- 3.18.** The following parameters were computed for a stream near Dallas, Texas, for 1940–1959, inclusive. The data were transformed to $\log_{10} Q = y$.

$$\bar{y} = 3.52 \quad (\text{mean}),$$

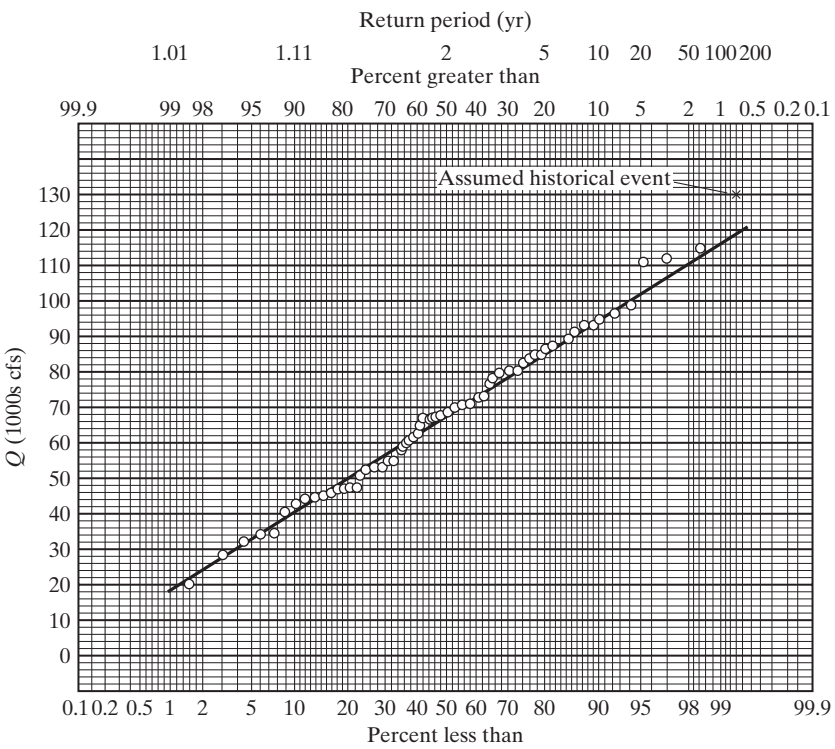
$$S_y = 0.50 \quad (\text{standard deviation}),$$

$$C_s = 0.50 \quad (\text{weighted skewness}).$$

Find the magnitude of the 50-yr flood, assuming that the annual peak flow follows (a) log Pearson 3 distribution and (b) lognormal distribution.

- 3.19.** A probability plot of 66 yr of peak discharges for the Kentucky River near Salvisa, Kentucky, is shown in Figure P3-19.
- (a) What probability distribution is being used?
 - (b) What are the mean and standard deviation of the peak discharges?
 - (c) If the distribution has other parameters, what are their values?
 - (d) What is the 50-yr flow?
 - (e) What is the 75-yr flow?
 - (f) What is the probability that the annual peak flow will be greater than or equal to 50,000 cfs for all of the next consecutive three years?
 - (g) What is the probability that at least one 100-yr event will occur in the next 40 years? In the next 100 years?
 - (h) Which plotting position has been used to plot the data points?
 - (i) Do the Kentucky River data appear to be skewed?
- 3.20.** A probability plot of 19 yr of peak discharges for the West Branch of the Mahoning River near Newton Falls, Ohio, is shown in Figure P3-20. This is an example of Gumbel, or extreme-value I, probability paper. Simply use the straight, fitted line to obtain the answers.
- (a) What is the 30-yr flow?
 - (b) What is the 120-yr flow?

Figure P3-19
Normal probability
plot for Kentucky
River data. (From
Haan, 1977, p. 137.)



- (c) What is the return period of a flow of 5200 cfs?
- (d) What is the probability that the annual peak discharge will fall between 5000 and 7000 cfs?
- 3.21.** The following annual total rainfall data for Houston Intercontinental Airport were collected over a 21-yr period.

Year	Rainfall (in.)	Year	Rainfall (in.)	Year	Rainfall (in.)
1970	48.19	1977	34.94	1984	48.19
1971	37.83	1978	44.93	1985	49.14
1972	50.80	1979	58.97	1986	44.93
1973	70.16	1980	38.99	1987	40.60
1974	49.29	1981	55.98	1988	22.93
1975	50.97	1982	42.87	1989	52.73
1976	54.62	1983	53.21	1990	40.37

- (a) Compute the mean, variance, and the skewness coefficient (C_s).
- (b) Plot a histogram using 5-in. intervals.

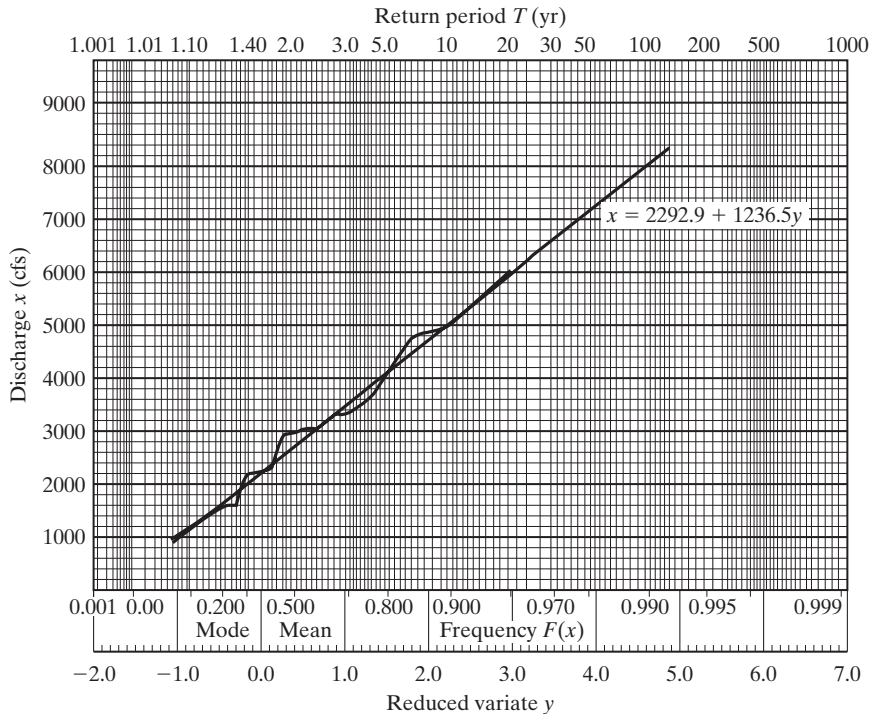


Figure P3-20

Annual floods of the West Branch Mahoning River near Newton Falls, Ohio, 1927–1945, on Gumbel probability paper. Vertical lines on the plot correspond to the lower probability scale, $F(x)$. (From National Bureau of Standards, 1953, *Probability Tables for the Analysis of Extreme Value Data*, Applied Mathematics Series 22, U.S. Government Printing Office, Washington, D.C.)

- (c) Fit the data with the normal distribution. Sketch the normal PDF on the histogram of part (b), scaling such that the areas under the histogram and under the PDF are the same (e.g., see Fig. 3–15).
 - (d) Find the value of the 10-yr annual rainfall total.
 - (e) Which years most closely represent the mean annual and 10-yr rainfalls for Houston?
- 3.22.** Explain how IDF curves (see Fig. 1–8) are statistically developed for any urban rainfall gage. Assume that data are available for 5-, 15-, 30-, and 60-min intervals up to 24 hr.
- 3.23.** Annual rainfall data for the Alvin, Texas, gage are given below. The data should be fitted using a log Pearson type 3 distribution. Decide if 1979 is an outlier by performing the analysis with and without the data point included.

Year	Rainfall (in.)	Year	Rainfall (in.)	Year	Rainfall (in.)
1970	48.82	1977	34.53	1984	45.99
1971	38.27	1978	41.43	1985	59.12
1972	53.34	1979	102.58	1986	51.75
1973	71.93	1980	41.15	1987	67.70
1974	51.85	1981	52.79	1988	34.19
1975	43.73	1982	42.89	1989	48.02
1976	54.52	1983	60.48	1990	41.45

To determine if a value is an outlier, perform the following analysis, as presented by the Interagency Advisory Committee on Water Data (1982): Determine the high and low outlier thresholds of the distribution. If an outlier occurs, then discard it from the dataset and repeat the analysis. These values can be calculated from the following equations:

$$y_H = \mu + K_n \sigma,$$

$$y_L = \mu - K_n \sigma,$$

where K_n is the one-sided 10% significance level for the normal distribution, a function of n . (For $n = 21$, $K_n = 2.408$. For $n = 20$, $K_n = 2.385$.) The value y_H is the high outlier threshold (in log units for the lognormal or LP3 distributions), y_L is the low outlier threshold (in log units for the lognormal or LP3 distributions), μ is the mean (of the log-transformed data for the lognormal or LP3 distributions), and σ is the standard deviation.

- 3.24.** The random variable x represents the depth of rainfall in June, July, and August in Houston. The whole PDF is *symmetric* and is shaped as an isosceles triangle, with base 0–60 in. Between values of $x = 0$ and $x = 30$, the probability density function has the equation

$$f(x) = \frac{x}{900}, \quad 0 \leq x \leq 30.$$

- Sketch the complete PDF. Demonstrate that $\int f(x) dx = 3.0$.
 - Find the probability that next summer's rainfall will not exceed 20 in.
 - Find the probability that summer rainfall will equal or exceed 30 in. for the next three consecutive summers.
 - For the above PDF, what is the mean value of summer rainfall?
- 3.25.** This problem asks you to perform a descriptive analysis using real data of interest to you. The problem should be done using spreadsheet or similar software.
- Download a series of annual maximum flows for a river of interest to you. USGS data may be obtained starting at the website <http://water.usgs.gov/nwis/>. Import the data into your spreadsheet or similar software. Convert the lines of text data into columnar data.
 - Note the characteristics of the basin from its description in the USGS files. What are the basin area and latitude and longitude of the gage? Are there diversions, controls, or storage (e.g., reservoirs) upstream?

- (c) Plot the time series of peak flows and $\log_{10}$ (flows) vs. water year. The series of $\log_{10}$ (flows) should have a lower coefficient of variation. Does the shape of the time-series plot of flows suggest that the time-series of river peak flows is nonstationary? If so, discuss possible reasons.
 - (d) Compute and plot relative-frequency histograms for the flows and $\log_{10}$ (flows). Discuss any difference in skewness evident from the two plots.
 - (e) Compute the following statistics for the series of flows and for the series of $\log_{10}$ (flows): number, average, unbiased variance, unbiased standard deviation, coefficient of variation, unbiased skewness, maximum, and minimum.
- 3.26.** For the data of Problem 3.25, compute a weighted skew coefficient according to the Bulletin 17B method.
- 3.27.** For the data of Problem 3.25, fit a log Pearson 3 distribution to the peak flows, using the method of moments described in this text. Use the weighted skew coefficient computed in Problem 3.26.
- (a) Compute estimated flows for return periods listed in Table 3–4.
 - (b) Plot the fitted CDF on lognormal probability paper.
 - (c) Using the Cunnane plotting-position formula (with parameter $a = 0.4$), plot enough of the measured flows to provide a comparison similar to Figure 3–19. Discuss the fit.
- 3.28.** Repeat Problem 3.27 using the lognormal and three-parameter gamma distributions.
- 3.29.** For a station of interest to you, download 10 yr of daily average streamflow data from the USGS website <http://water.usgs.gov/nwis/>. Paste the data into a spreadsheet and convert the text data to columns. Construct and plot a flow-duration curve for these data. From the table and chart, what are the flows equaled or exceeded 20%, 50%, and 90% of the time?
- 3.30.** Interevent times for winter storms arriving at Corvallis, Oregon, for the months November through April for the winters of 1996–97, 1997–98, and 1998–99 were determined, and a frequency histogram prepared as shown in the table below. The average interevent time was 2.59 days.
- (a) Fit an exponential distribution to these data by finding the parameter λ .
 - (b) Plot the relative-frequency histogram and the fitted exponential PDF on the same chart. Care may need to be taken to be sure that the histogram and PDF are properly aligned. Each day (0–1, 1–2, and so on) is a class interval.
 - (c) From the relative-frequency histogram, compute the cumulative-frequency histogram and plot on arithmetic graph paper.
 - (d) On two-cycle semilog paper (or using spreadsheet options for log-scales), plot the empirical CDF from part (c) and the fitted CDF. On this “probability paper,” values should be plotted at the class mark, centering on

half-days. The empirical values from part (c) should be plotted as individual points, and one fitted CDF (exceedance probability) should be plotted as a straight line.

- (e) What is the probability that the time between winter storms is ≤ 3 days? Compute using both the empirical CDF and the fitted CDF.

Interevent Time (days)	Frequency
0–1	35
1–2	12
2–3	6
3–4	6
4–5	7
5–6	3
6–7	1
7–8	1
8–9	3
9–10	2
10–11	1
11–12	0
12–13	0
13–14	1
>14	0

- 3.31.** The data presented in the table for Problem 3.30 are known as **grouped data**, of the type that are developed in order to plot a frequency histogram. The mean of such data can be determined as a weighted average of the class marks, as follows:

$$\bar{t} = \frac{\sum_{i=1}^k f_i t_i}{\sum_{i=1}^k f_i}$$

where f_i and t_i are the frequency and class mark, respectively, for k class intervals.

- (a) Demonstrate that the mean of the interevent times of Problem 3.30 is as stated.
- (b) How many interevent time values were used in the analysis?
- 3.32.** Using statistics for Siletz River peak flows, 1979–1999 (Table 3–1), generate a series of normally distributed synthetic streamflows by following these guidelines.
- (a) By performing a regression of flows for the period 1980–1999 vs. flows during 1979–1998, verify that the serial correlation coefficient for this time period is 0.1411.
- (b) Verify that the mean and unbiased standard deviation for the full 21-year period are 19,343 cfs and 7217 cfs, respectively. Use these values for part (c).

- (c) The list of 21 $N(0, 1)$ random numbers below was generated in Excel using the Tools/Data Analysis/Random Number Generation option with a seed of 12345. (The option for a seed allows one to generate identical sequences of random numbers.) Assuming that the initial flow = mean (at “step 0”), generate a sequence of 21 random flows using Equation (3–83). Compute the mean, standard deviation, and serial correlation coefficient of the synthetic flow sequence to see how well these statistics are preserved. *Optional:* Create a new series of $N(0, 1)$ random numbers and repeat the generation. Notice that as the mean and standard deviation of the random numbers differ from 0 and 1, respectively, so do the mean and standard deviation of the synthetic sequence differ from their historic values.

n	z	n	z	n	z
0	n/a				
1	-0.7341	8	-0.9733	15	-0.5506
2	0.2143	9	1.2119	16	-0.1774
3	0.7968	10	0.5659	17	0.4409
4	0.4544	11	-0.1092	18	-0.4908
5	-0.9235	12	-0.1214	19	0.8266
6	0.5659	13	0.3157	20	-1.1724
7	0.9885	14	0.4213	21	-0.4906

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Chapter 4

Flood Routing



Mansfield Dam, Hill Country of Texas.

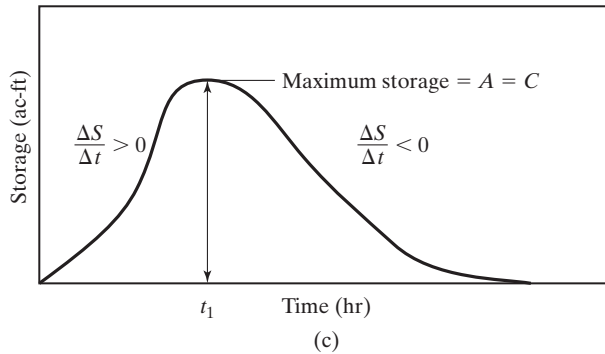
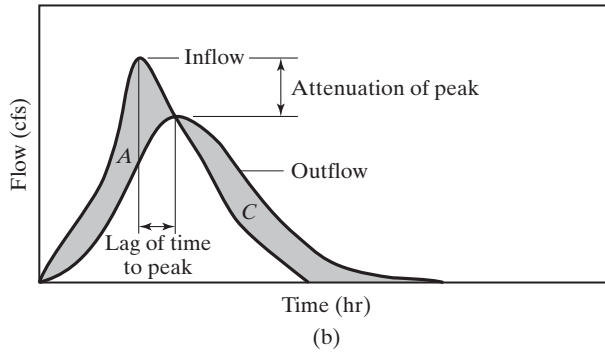
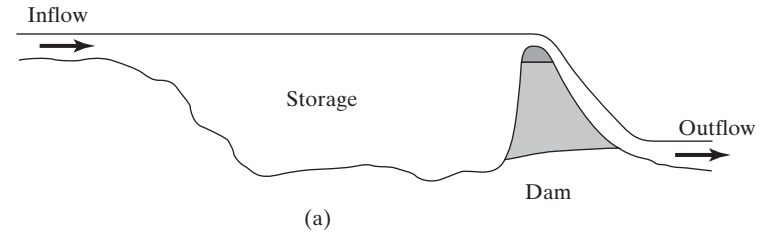
The movement of a flood wave down a channel or through a reservoir and the associated change in timing or attenuation of the wave constitute an important topic in floodplain hydrology. It is essential to understand the theoretical and practical aspects of flood routing to predict the temporal and spatial variations of a flood wave through a river reach or reservoir. Flood routing methods can also be used to predict the outflow hydrograph from a watershed subjected to a known amount of precipitation.

The **storage routing** concept is most easily understood by referring to Figure 4–1. Inflow and outflow hydrographs for a small level-surface reservoir have been plotted on the same graph. Area *A* represents the volume of water that fills available storage up to time t_1 . Inflow exceeds outflow and the reservoir is filling. At time t_1 , inflow and outflow are equal and the maximum storage is reached. For times exceeding t_1 , outflow exceeds inflow and the reservoir empties. Area *C* represents the volume of water that flows out of the reservoir and must equal area *A* if the reservoir begins and ends at the same level. The peak of the outflow from a reservoir should intersect the inflow hydrograph as shown in Figure 4–1, since, in general, the outflow is uniquely determined by reservoir storage or level.

4.1 HYDROLOGIC AND HYDRAULIC ROUTING

Figure 4-1

Reservoir concepts.
 (a) Reservoir storage.
 (b) Inflow to and outflow from the reservoir.
 (c) Storage in the reservoir.



We will see that storage routing through a reservoir will generally attenuate the peak outflow and lag the time to peak for the outflow hydrograph. The rate of change of storage can be written as the continuity equation

$$I - Q = \frac{\Delta S}{\Delta t}, \quad (4-1)$$

where

I = inflow,
 Q = outflow,
 ΔS = change in storage,
 Δt = change in time.

Example 4-1 is a detailed illustration of reservoir storage concepts.

STORAGE COMPUTATIONS

EXAMPLE 4-1

Inflow and outflow hydrographs for a reservoir are depicted in Figure E4-1(a).

- Determine the average storage for each one-day period ($\Delta t = 1$ day). Graph storage vs. time for the reservoir for the event. Assume that $S_0 = 0$ (the reservoir is initially empty).
- What is the (approximate) maximum storage reached during this storm event?

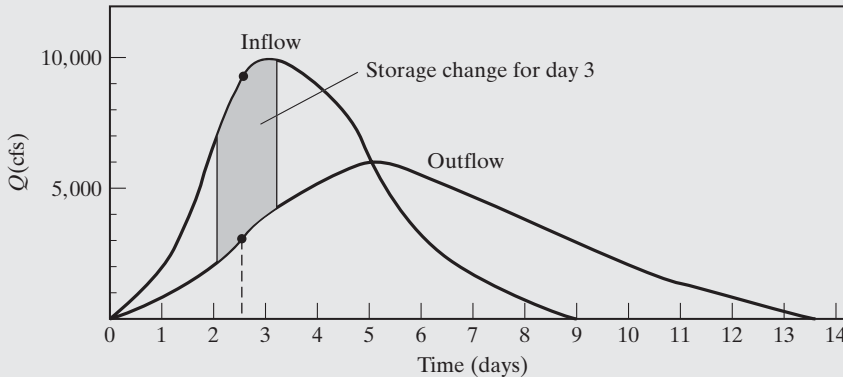


Figure E4-1(a)

- The rate of change in storage is equal to inflow minus outflow. First, we tabulate values of I and Q and take their difference. Storage is equal to the area between the inflow and outflow curves, or

$$S = \int (I - Q) dt.$$

This integral can be simply approximated by

$$S = \sum_i (\bar{I} - \bar{Q}) \Delta t,$$

where $\bar{I}$ and $\bar{Q}$ are averages for each day. Using a simple approach, $\bar{I}$ and $\bar{Q}$ values are averaged at noon each day. Other more accurate methods could be used for the numerical integration and include the trapezoidal rule, or Simpson's rule (Chapra and Canale, 2006). Any of these methods can be used as in Example 2-1 to determine volumes under hydrographs.

SOLUTION

Time (days)	I -(cfs)	Q -(cfs)	$\Delta S/\Delta t$ (cfs)
0.5	500	250	250
1.5	3500	1000	2500
2.5	9000	3000	6000
3.5	9750	4500	5250
4.5	8000	5750	2250
5.5	4500	6000	-1500
6.5	2250	5250	-3000
7.5	1250	4250	-3000
8.5	250	3250	-3000
9.5	0	2500	-2500
10.5	0	1500	-1500
11.5	0	1000	-1000
12.5	0	750	-750
13.5	0	0	0

Using $\Delta t = 1$ day, storage at the end of the first day, S_1 , is

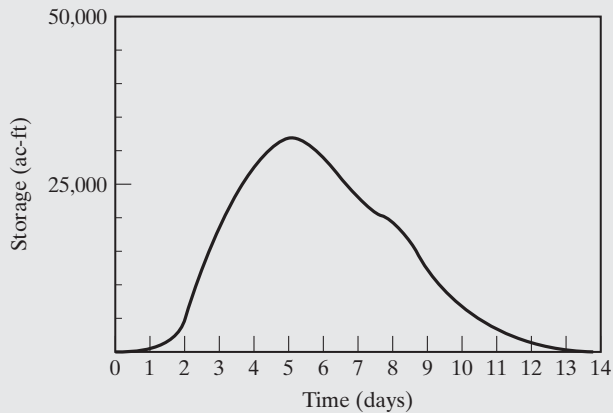
$$\begin{aligned}
 S_1 &= S_0 + (\bar{I}_1 - \bar{Q}_1) \Delta t \\
 &= 0 + (250 \text{ cfs})(1 \text{ day}) \left(\frac{24 \text{ hr}}{\text{day}} \right) \left(\frac{3600 \text{ s}}{\text{hr}} \right) \left(\frac{\text{ac}}{43,560 \text{ ft}^2} \right) \\
 &= 496 \text{ ac-ft.}
 \end{aligned}$$

For day 2, cumulative storage becomes

$$\begin{aligned}
 S_2 &= S_1 + (\bar{I}_2 - \bar{Q}_2) \Delta t, \\
 S_2 &= 496 + (2500)(24)(3600) \left(\frac{1}{43,560} \right) \text{ ac-ft} \\
 &= 5455 \text{ ac-ft.}
 \end{aligned}$$

The procedure is shown completed in the following table and the storage curve in Figure E4-1(b).

Figure E4-1(b)



Time (day)	Storage (ac-ft)
1	496
2	5455
3	17,356
4	27,769
5	32,232
6	29,256
7	23,306
8	17,356
9	11,405
10	6446
11	3471
12	1488
13	0
14	0

- (b) The maximum storage, as seen from the table and figure, is 32,232 ac-ft. This occurs at day 5 for this event, as seen from the equation

$$\frac{ds}{dt} = I - Q.$$

$S_{\max}$ will occur when dS/dt equals zero. At this point, $I = Q$, which occurs at day 5 on the inflow–outflow hydrographs.

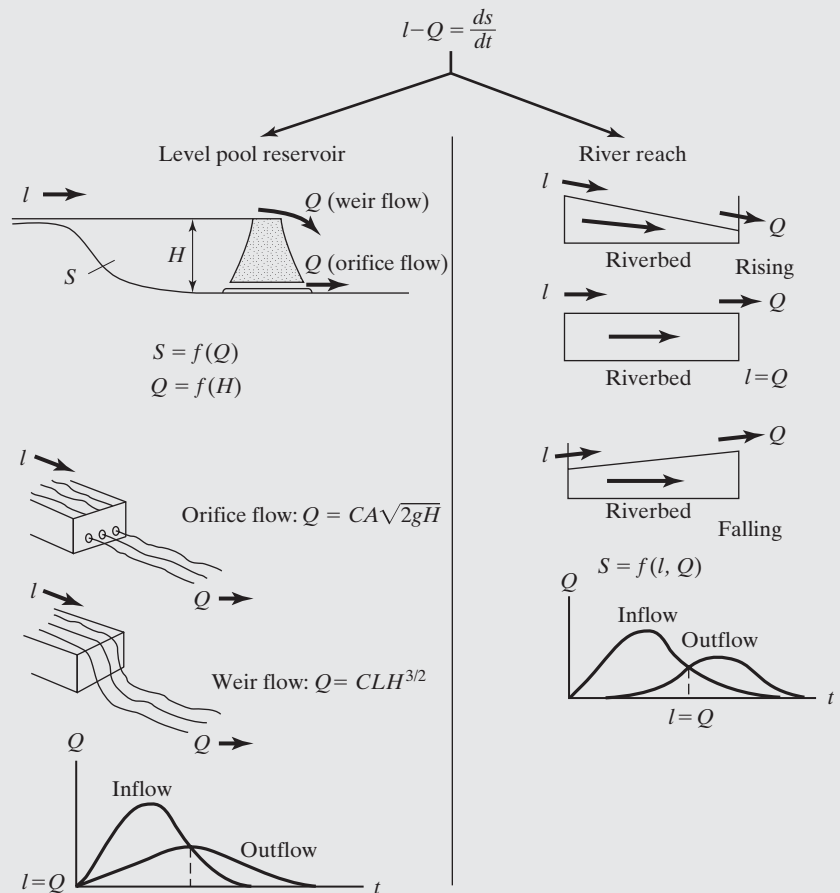
River routing differs from reservoir routing in that storage in a river reach of length L depends on more than just outflow. The peak of the outflow hydrograph from a reach is usually attenuated and delayed compared with that of the inflow hydrograph. Because storage in a river reach is a function of whether stages are rising or falling, storage in this case is a function of both outflow and inflow for the routing reach (see Section 4.3). Also, as river stages rise high enough to inundate a floodplain beyond the banks of the channel, significant velocity reductions are observed in the floodplain compared with the main channel. Example 4–2 presents differences between river and reservoir routing.

RIVER AND RESERVOIR ROUTING CONCEPTS

EXAMPLE 4–2

Figure E4–2(a) illustrates some differences between river and reservoir routing. Prove that, for a level-pool reservoir, the peak of the outflow hydrograph must intersect the inflow hydrograph.

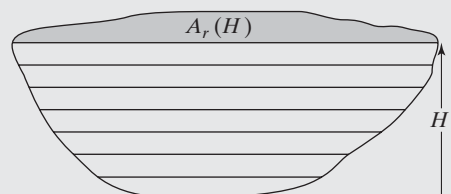
Figure E4-2(a)

**SOLUTION**

Storage in a reservoir may be determined from the height of water in the reservoir [see Fig. E4-2(b)]. For instance, in a level-pool reservoir, A is a function of depth such that

$$S = f(H) = \int A_r(H) dH,$$

Figure E4-2(b)



and

$$A_r = dS/dH.$$

While inflow increases storage in a level-pool reservoir, outflow may be determined, whether or not inflow is known, if the storage is known:

$$Q = f(S)$$

but

$$S = f(H),$$

and, therefore,

$$Q = f(H).$$

From the equation of continuity,

$$I - Q = \frac{dS}{dt} = A_r(H) \frac{dH}{dt},$$

$$I = Q \quad \text{when} \quad \frac{dS}{dt} = 0.$$

Since S is directly proportional to H ,

$$\frac{dH}{dt} = 0 \quad \text{when} \quad \frac{dS}{dt} = 0,$$

and since Q is also directly proportional to H ,

$$\frac{dQ}{dt} = 0 \quad \text{when} \quad \frac{dH}{dt} = 0.$$

Thus,

$$\frac{dQ}{dt} = 0 \quad \text{when} \quad \frac{dS}{dt} = 0.$$

This occurs when Q and S are maximum, or at $I = Q$ [Fig. E4-2(a)].

Hydrologic Routing Methods

Routing techniques may be classified in two major categories: simple **hydrologic routing** and more complex **hydraulic routing**. Hydrologic routing involves the balancing of inflow, outflow, and volume of storage through the use of the continuity equation. A second relationship, the storage–discharge relation, is also required between outflow rate and storage in the system. Applications of hydrologic routing techniques to problems of flood prediction, flood control measures, reservoir design and operation, watershed simulation, and urban design are numerous. Many computer models are

available that take input rainfall, convert it to outflow hydrographs, and then route the hydrographs through complex river or reservoir networks using hydrologic routing methods. These applications are presented in detail in Chapters 5 and 6.

Hydraulic Routing Methods

Hydraulic routing is more complex and accurate than hydrologic routing. It is based on the solution of the continuity equation and the momentum equation for unsteady flow in open channels. Usually solved by explicit or implicit numerical methods on a computer, these differential equations are known as the Saint Venant equations, first derived in 1871, for which no closed-form solutions exist.

Unsteady flow in rivers, reservoirs, and estuaries is caused by motion of long waves due to tides, flood waves, storm surges, and dynamic reservoir releases. These types of wave forms can be adequately described only by the one-dimensional Saint Venant equations, which are presented in detail in Section 4.4. In many cases, the governing equations can be simplified to a one-dimensional continuity equation and a uniform flow relationship, referred to as **kinematic wave routing**, which implies that discharge can be computed as a simple function of depth alone. In 1981, kinematic wave routing was added to the HEC-HMS flood hydrograph package with a numerical solver, as described in Section 4.6.

Uniform flow implies a balance between gravitational and frictional forces in the channel. This assumption cannot always be justified, especially on very flat slopes where effects of water surface slope cannot be ignored. Cases where other terms in the momentum equation for hydraulic routing must be retained include (1) upstream movement of tides and storm surges, (2) backwater effects from downstream reservoirs and tributary inflows, (3) flood waves in channels of very flat slope (2–3 ft/mi), and (4) abrupt waves caused by sudden releases from reservoirs or dam failures. For these cases, the complete solution of the Saint Venant equations should be used. Several computer models exist to solve these equations.

4.2 HYDROLOGIC RIVER ROUTING

As a flood wave passes through a river reach, the peak of the outflow hydrograph is usually attenuated and delayed due to channel resistance and storage capacity. Considering a lumped storage approach for the reach, the difference between the ordinates of the inflow and outflow hydrographs, represented by shaded areas in Figure 4–2, is equal to the rate of change of storage in the reach, as shown in Equation (4–1). The value of $\Delta S/\Delta t$ in the continuity equation is positive when storage is increasing and negative when storage is decreasing, and S can be plotted as a function of time. Equation (4–1) can be written in finite-difference

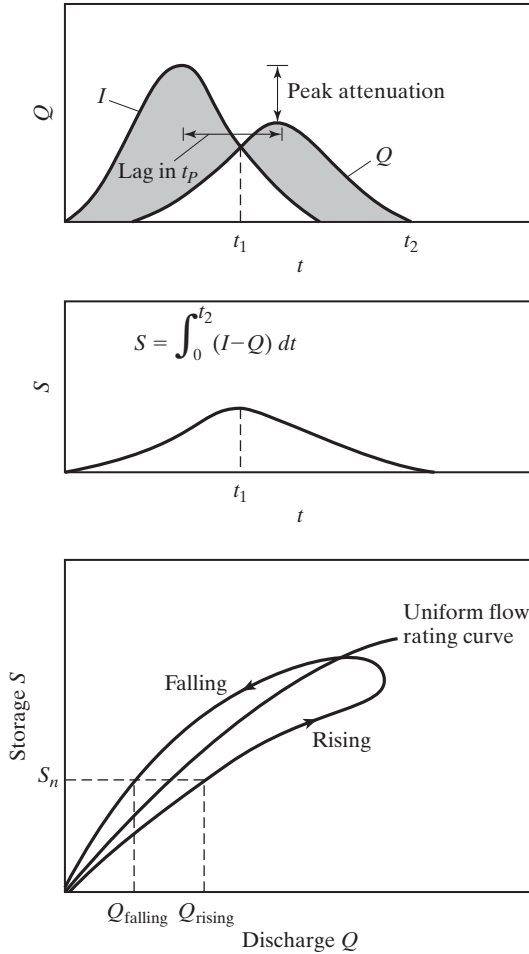


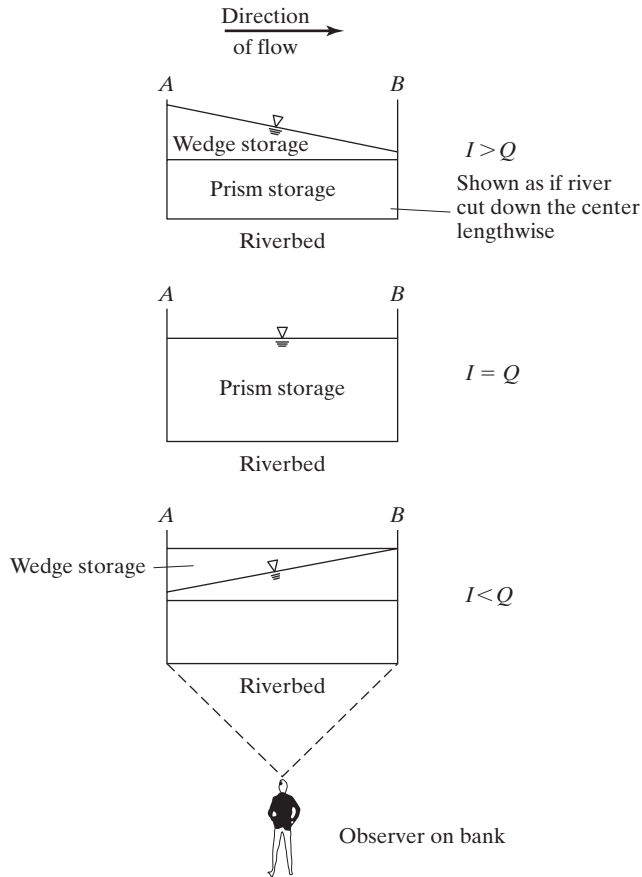
Figure 4-2
Storage in a river reach.

form as Equation (4-2), where Δt is referred to as the routing time period and subscripts 1 and 2 denote the beginning and end of the time period, respectively:

$$\frac{1}{2}(I_1 + I_2) - \frac{1}{2}(Q_1 + Q_2) = \frac{S_2 - S_1}{\Delta t}. \quad (4-2)$$

If storage is plotted against outflow for a river reach, the resulting curve will generally take the form of a loop, as shown in Figure 4-2. This loop effect implies greater storage for a given outflow during falling stages than during rising stages. If one considers water surface profiles at various times during the passage of the flood wave, the concept of prism and wedge storage is useful. This is shown in Figure 4-3. A large volume of wedge storage may exist during rising stages before outflows have increased. During falling stages,

Figure 4-3
Prism and wedge
storage concepts.



inflow drops more rapidly than outflow, and the wedge storage becomes negative. Hydrologic routing in rivers and channels thus requires a storage relationship that allows for wedge storage. This is accomplished by allowing storage to be a function of both inflow and outflow, as in the Muskingum method of flood routing (McCarthy, 1938). The method suffers the disadvantage of assuming the uniform flow rating curve in place of the loop curve shown in Figure 4-2.

Muskingum Method

The Muskingum method was developed by McCarthy (1938) and utilizes the continuity equation [Eq. (4-2)] and a storage relationship that depends on both inflow and outflow. The storage within the reach at a given time can be related to inflow and outflow by (Chow, 1959)

$$S = \frac{b[xI^{m/n} + (1-x)Q^{m/n}]}{a^{m/n}}, \quad (4-3)$$

where inflow I and outflow Q are related to ay^n from Manning's equation, where a and n are constants. Storage in the reach is related to by^m , where b and m are constants that depend on the cross section of the channel. The parameter x defines the relative weighting of inflow and outflow in determining storage volume in the reach.

The Muskingum method assumes that flow and storage are both related to depth such that $m/n = 1$ and $b/a = K$, the travel time, resulting in a linear relationship of the form

$$S = K[xI + (1 - x)Q], \quad (4-4)$$

where

K = travel time constant for the reach,

x = weighting factor, which varies from 0 to 0.5 for a given reach.

For the case of linear reservoir routing where S depends only on outflow, $x = 0$ in Equation (4-4). In smooth uniform channels, $x = 0.5$ yields equal weight to inflow and outflow, which theoretically results in pure translation of the wave. A typical range for most natural streams is $x = 0.2$ to 0.3 , which results in some attenuation of the flood wave.

The routing procedure uses the finite-difference form of continuity Equation (4-2) combined with Equation (4-4) in the form

$$S_2 - S_1 = K[x(I_2 - I_1) + (1 - x)(Q_2 - Q_1)] \quad (4-5)$$

to produce the Muskingum routing equation for a river reach:

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1, \quad (4-6)$$

where

$$C_0 = \frac{-Kx + 0.5 \Delta t}{D}, \quad (4-7)$$

$$C_1 = \frac{Kx + 0.5 \Delta t}{D}, \quad (4-8)$$

$$C_2 = \frac{K - Kx - 0.5 \Delta t}{D}, \quad (4-9)$$

$$D = K - Kx + 0.5 \Delta t. \quad (4-10)$$

This procedure is ideally set up for a calculator or a personal computer. Note that K and Δt must have the same units, and $2Kx < \Delta t \leq K$ for numerical accuracy, and that the coefficients C_0 , C_1 , and C_2 sum to 1.0. The routing operation is accomplished by solving Equation (4-6) for successive time increments, with Q_2 of one routing period becoming Q_1 of the succeeding period. Example 4-3 illustrates the row-by-row computation, and a spreadsheet program can be easily written in Excel.

EXAMPLE 4-3**MUSKINGUM ROUTING**

Route the inflow hydrograph tabulated in the following table through a river reach for which $x = 0.2$ and $K = 2$ days. Use a routing period $\Delta t = 1$ day and assume that inflow equals outflow for the first day.

Time (day)	Inflow (cfs)
1	4000
2	7000
3	11,000
4	17,000
5	22,000
6	27,000
7	30,000
8	28,000
9	25,000
10	23,000
11	20,000
12	17,000
13	14,000
14	11,000
15	8000
16	5000
17	4000
18	4000
19	4000
20	4000

SOLUTION

First, we determine the coefficients C_0 , C_1 , and C_2 for the reach:

$$C_0 = \frac{-Kx + 0.5 \Delta t}{D},$$

$$C_1 = \frac{Kx + 0.5 \Delta t}{D},$$

$$C_2 = \frac{K - Kx - 0.5 \Delta t}{D},$$

$$D = K - Kx + 0.5 \Delta t.$$

For $K = 2$ days, $\Delta t = 1$ day, and $x = 0.2$,

$$\begin{aligned} D &= 2 - 2(0.2) + 0.5(1) \\ &= 2.1, \end{aligned}$$

$$C_0 = \frac{-(2)(0.2) + (0.5)(1)}{(2.1)},$$

$$C_0 = 0.0476,$$

$$C_1 = 0.4286,$$

$$C_2 = 0.5238.$$

We may check our computations by seeing if the coefficients sum to 1:

$$(0.0476) + (0.4286) + (0.5238) = 1.0000.$$

We substitute these values into Equation (4-6) to obtain

$$Q_2 = (0.0476)I_2 + (0.4286)I_1 + (0.5238)Q_1.$$

For $t = 1$ day,

$$Q_1 = I_1 = 4000 \text{ cfs.}$$

For $t = 2$ days,

$$\begin{aligned} Q_2 &= (0.0476)(7000) + (0.4286)(4000) + (0.5238)(4000) \\ &= 4143 \text{ cfs.} \end{aligned}$$

For $t = 3$ days,

$$\begin{aligned} Q_3 &= (0.0476)(11,000) + (0.4286)(7000) + (0.5238)(4143) \\ &= 5694 \text{ cfs.} \end{aligned}$$

This procedure is shown completed for $t = 1$ to $t = 20$ days in the following table.

Time	Inflow (cfs)	Outflow (cfs)
1	4000	4000
2	7000	4143
3	11,000	5694
4	17,000	8506
5	22,000	12,789
6	27,000	17,413
7	$I_p \rightarrow 30,000$	22,121
8	28,000	25,778
9	25,000	26,693 $\leftarrow Q_p$
10	23,000	25,792
11	20,000	24,319
12	17,000	22,120
13	14,000	19,539
14	11,000	16,758
15	8000	13,873
16	5000	10,934
17	4000	8061
18	4000	6127
19	4000	5114
20	4000	4583

Note: Q_p lags I_p by two days, i.e., approximately by K .

It is possible to compute outflow at any time if the following inflows are known: $I_1, I_2, I_3, \dots, I_n$. Equation (4-6) can be rewritten as

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1} \quad (4-11)$$

and

$$Q_{n-1} = C_0 I_{n-1} + C_1 I_{n-2} + C_2 Q_{n-2}.$$

Repeated calculations for $Q_{n-2}, Q_{n-3}, \dots$ can be performed so that the following equation for Q_n can be derived:

$$Q_n = K_1 I_n + K_2 I_{n-1} + K_3 I_{n-2} + \dots + K_n I_1, \quad (4-12)$$

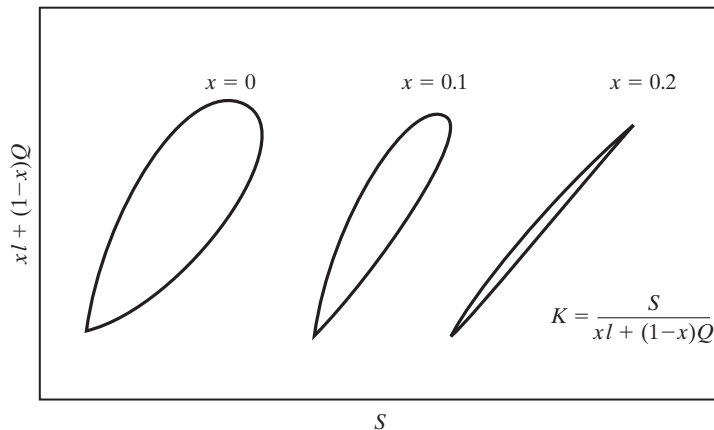
where

$$\begin{aligned} K_1 &= C_0; & K_2 &= C_0 C_2 + C_1; & K_3 &= K_2 C_2, \\ K_i &= K_{i-1} C_2 & \text{for } i &> 2. \end{aligned}$$

Determination of Storage Constants

The Muskingum K is usually estimated from the travel time for a flood wave through the reach, and x averages 0.2 to 0.3 for a natural stream. However, if both inflow and outflow hydrograph records are available, better estimates for K and x can be made through graphical methods. Storage S is plotted vs. weighted discharge, $xI + (1-x)Q$, for several selected values of x , and the plot that yields the most linear single-valued curve provides the best value for x . The Muskingum method assumes that this curve is a straight line with reciprocal slope K . Figure 4-4 and Example 4-4 illustrate the concept of selecting x and K . Thus, the Muskingum method assumes that storage is a single-valued function of weighted inflow and outflow. Normally, a river must be divided into several reaches for application of the Muskingum routing method in order to maintain numerical stability. This requires that the flow changes slowly with time. This method has been shown to work quite

Figure 4-4
Selection of Muskingum coefficients.



well for ordinary streams with small slopes where the storage–discharge curve is approximately linear. However, in cases involving very steep or mild slopes, backwater effects, or abrupt waves, dynamic effects of flow may be pronounced and hydraulic routing methods should be used rather than hydrologic methods. Alternatively, the Muskingum–Cunge method may be used as a more accurate version of the Muskingum method. Hydraulic routing methods are described in Section 4.7.

DETERMINATION OF THE MUSKINGUM ROUTING COEFFICIENTS

EXAMPLE 4–4

The values listed in Table E4–4 for inflow, outflow, and storage were measured for a particular reach of a river. Determine the coefficients K and x for use in the Muskingum routing equations for this reach.

Table E4–4

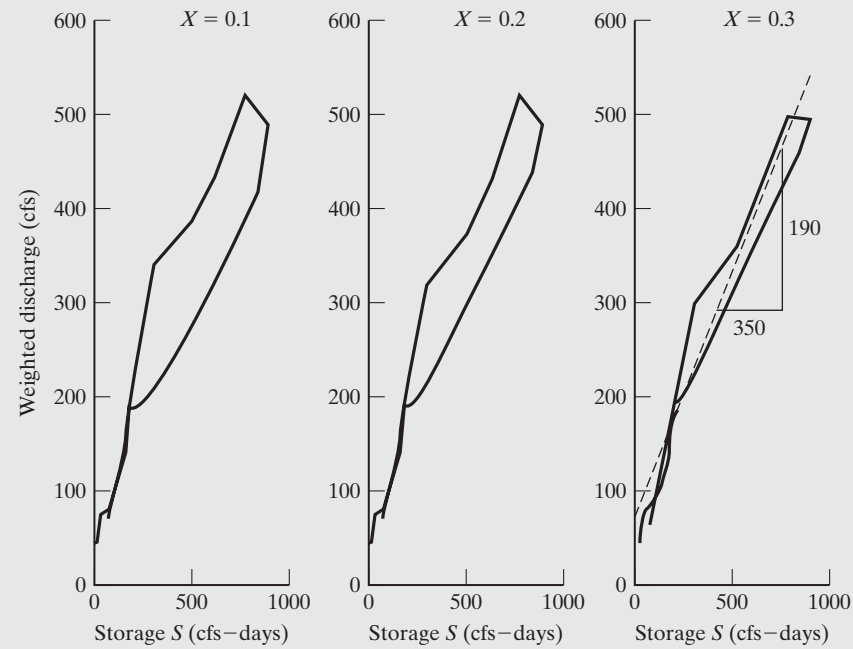
Time (days)	Avg. Inflow (cfs)	Avg. Outflow (cfs)	Storage (cfs-days)
1	59	42	17
2	93	70	40
3	129	76	94
4	205	142	157
5	210	183	184
6	234	185	233
7	325	213	345
8	554	293	606
9	627	397	836
10	526	487	875
11	432	533	774
12	400	487	687
13	388	446	629
14	270	400	499
15	162	360	301
16	124	230	195
17	102	140	157
18	81	115	123
19	60	93	90
20	51	71	70

To determine Muskingum coefficients, we guess a value of x and then plot $[xI + (1 - x)Q]$ vs. S . The one plot that comes closest to being a straight line is chosen to determine the coefficient values. The average value is $x = 0.2$ for a natural stream. Therefore, we assume that x must lie between 0.1 and 0.3. Plots of $[xI + (1 - x)Q]$ vs. S are made for $x = 0.1$, $x = 0.2$, and $x = 0.3$ using the values listed in Table E4–4 (see Fig. E4–4).

SOLUTION

Storage (cfs-days)	$[xI + (1 - x)Q]$ (cfs)		
	$x = 0.1$	$x = 0.2$	$x = 0.3$
17	43	45	47
40	72	74	77
94	81	86	92
157	148	155	161
184	186	188	191
233	190	195	200
345	224	235	247
606	319	345	371
836	420	443	466
875	491	495	499
774	523	513	503
687	478	470	461
629	440	434	429
499	387	374	361
301	340	320	301
195	219	209	198
157	136	132	129
123	112	108	105
90	89	86	83
70	69	67	65

Figure E4-4



It can easily be seen that the plot for $x = 0.3$ is the straightest line. K is calculated as the inverse slope of the line:

$$1/K = \frac{490 \text{ cfs} - 300 \text{ cfs}}{750 \text{ cfs} - \text{day} - 400 \text{ cfs} - \text{day}} = 0.543,$$

$$K = 1.8 \text{ days}.$$

For most streams, there will be some looping effects for various values of x . The most linear relationship is chosen to determine x and K .

Storage-Indication Method

Reservoir or detention basin routing is generally easier to perform than river routing because storage–discharge relations for pipes, weirs, and spillways are single-valued functions independent of inflow. Thus, a simple **storage-indication method** or **modified Puls method** uses the finite-difference form of the continuity equation combined with a storage-indication curve ($2S/\Delta t + Q$ vs. Q). Equation (4–2) can be generalized to the following finite-difference equation for two points in time:

$$(I_n + I_{n+1}) + \left(\frac{2S_n}{\Delta t} - Q_n \right) = \left(\frac{2S_{n+1}}{\Delta t} + Q_{n+1} \right), \quad (4-13)$$

in which the only unknowns are S_{n+1} and Q_{n+1} on the right-hand side. I is known for all n , and S_n and Q_n are known for the initial time step; therefore, the right-hand side of Equation (4–13) can be calculated. Values of S_{n+1} and Q_{n+1} are then used as input on the left-hand side, and the calculation is repeated for the second time interval, and so on. The storage-indication curve is a plot of $2S/\Delta t + Q$ vs. Q , as shown in Example 4–5. Thus, once the right-hand side of Equation (4–13) has been determined, one can read values of Q directly from the curve. Values for $2S/\Delta t - Q$ for the left-hand side of Equation (4–13) are calculated by subtracting $2(Q)$ from the right-hand side values. The detailed computations are shown in Example 4–5. Other storage reservoir routing examples are included in Chapters 5, 6, and 9.

4.3 HYDROLOGIC RESERVOIR ROUTING

STORAGE-INDICATION ROUTING

EXAMPLE 4–5

The design inflow hydrograph is shown in Figure E4–5(a), developed for a commercial area, and is to be routed through the reservoir (pond) shown in Figure E4–5(b). Initial conditions must be known; assume that initially the reservoir is empty ($S_0 = 0$) and there is no initial outflow ($Q_0 = 0$). The reservoir has a trapezoidal plan view, with side slope of 4:1 (horizontal: vertical) up to depth 5 ft above the bottom, and 2:1 between 5 and 7 ft. The crest of the emergency spillway is at 5 ft above the bottom of the pond.

Figure E4-5(a)
Inflow and outflow
hydrographs.

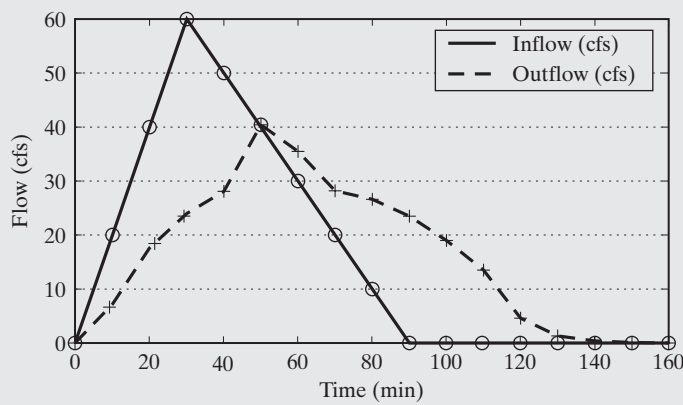
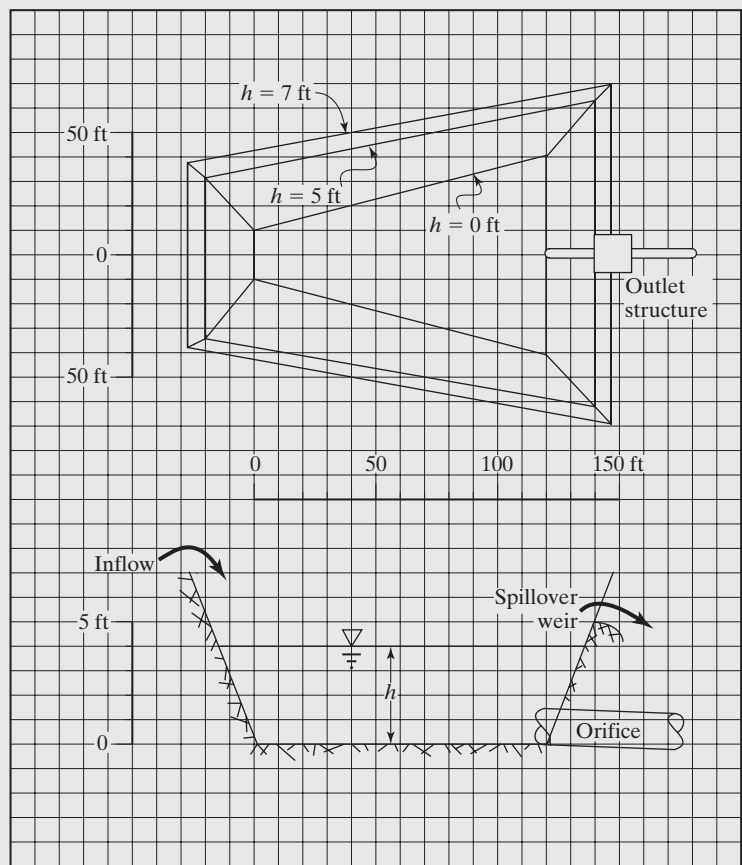


Figure E4-5(b)
Pond details.



The lower outflow is an 18-in. reinforced concrete pipe (RCP) that will be assumed to behave as an orifice with an entrance loss coefficient, C_d , of 0.9:

$$Q_{\text{orifice}} = C_d A_o \sqrt{2gh},$$

where

- Q_{orifice} = orifice outflow (cfs or cms),
- C_d = orifice entrance loss coefficient,
- A_o = orifice area = $\pi D^2/4$ for a circular pipe of diameter D ,
- g = gravitational acceleration (32.2 ft/s² or 9.8 m/s²), and
- h = depth above the orifice centerline (ft or m).

The ogee spillway is assumed to behave like a weir (Davis, 1952; French, 1985) with equation

$$Q_{\text{weir}} = C_e (2/3) \sqrt{2g} L (h - h_0)^{1.5},$$

where

- Q_{weir} = spillway discharge (cfs or cms),
- C_e = weir discharge coefficient,
- L = weir length (perpendicular to discharge) (ft or m), and
- h_0 = elevation of weir crest (ft or m).

For this example, $D = 1.5$ ft, $C_d = 0.9$, $C_e \approx 0.7$ (Davis, 1952), $L = 15$ ft, and $h_0 = 5$ ft. The orifice and weir outflows are assumed to obey the indicated equations even for low heads. The stage–discharge analysis could be refined in the event that this assumption must be modified. The sum of the orifice and weir outflows constitutes a **rating curve** for the small reservoir. The rating curve could be tabular or derived from experiments in lieu of equations such as those for the orifice and weir.

Using the depth, storage, and outflow relationships developed in the table, route the hydrograph through the reservoir. What is the maximum height reached in the reservoir for this inflow? Use $\Delta t = 10$ min.

h (ft)	Trap Bottom (ft)	Trap Top (ft)	Trap Height (ft)	Trap Area (ft ²)	Increm. Vol. (ft ³)	Cum. Vol. (ft ³)	Q_{orifice} (cfs)	Q_{weir} (cfs)	Q_{total} (cfs)	$2S/\Delta t + Q$ (cfs)
0	80.0	20.0	120	6000	0	0	0	0	0	0
1	88.9	28.9	128	7545	6772	6772	12.8	0	13	35
2	97.9	37.9	136	9233	8389	15,161	18.0	0	18	69
3	106.8	46.8	144	11,064	10,148	25,310	22.1	0	22	106
4	115.8	55.8	152	13,038	12,051	37,361	25.5	0	26	150
5	124.7	64.7	160	15,155	14,097	51,457	28.5	0	29	200
5.5	127.0	67.0	162	15,707	7716	59,173	29.9	19.9	50	247
6	129.2	69.2	164	16,268	7994	67,167	31.3	56.2	87	311
6.5	131.4	71.4	166	16,837	8276	75,443	32.5	103.2	136	387
7	133.7	73.7	168	17,416	8563	84,006	33.8	158.9	193	473

SOLUTION

There are two steps to the solution: In step 1, we develop a storage-indication curve from the indicated geometry and outflow relationships. This will involve additional computations to determine storage vs. depth. Step 2 is the routing procedure using the continuity equation and the storage-indication curve.

The geometry of the trapezoids results in the surface areas shown in the table. Storage (volume) S is the integral of surface area A :

$$S = \int A \, dh.$$

The integration is performed numerically in the table, using the **trapezoidal rule**. In this case, incremental volumes are computed by assuming the function $A(h)$ can be approximated as a series of trapezoids with sides A_i and A_{i+1} and width Δh . Then the incremental volume is

$$\Delta S = \frac{A_i + A_{i+1}}{2} \Delta h.$$

Thus, the incremental volume between depth 2 ft and 3 ft is

$$\Delta S = \frac{9233 \text{ ft}^2 + 11,064 \text{ ft}^2}{2} \cdot 1 \text{ ft} = 10,148 \text{ ft}^3.$$

Cumulative volume is simply the sum of the incremental volumes. Note that the elevation increment Δh need not remain constant and can be reduced if greater definition of outflows is needed—above the weir crest, for instance. Also, note that the depth h is an intermediate variable in the computations and is not used in the remainder of the procedure. Finally, observe the importance of accurate contours, normally derived from a survey, in developing the stage-vs.-volume relationship.

Next, we develop a storage-indication curve for the pond. This is a plot of $(2S/\Delta t) + Q$ vs. Q (where Q is total outflow). For instance, at $Q = 50$ cfs, $S = 59,173 \text{ ft}^3$ and

$$\frac{2S}{\Delta t} + Q = \frac{2(59,173 \text{ ft}^3)}{(10 \text{ min})(60 \text{ s/min})} + 50 \text{ cfs} = 247 \text{ cfs}.$$

These values are tabulated in the preceding table, and the storage-indication curve is plotted in Figure E4-5(c). The curve is simply another way of presenting the hydraulic relationship between storage and outflow. The meaning is that if by another mechanism the quantity $2S/\Delta t + Q$ can be determined, then the outflow Q can be determined from this relationship. Note that the storage-indication curve has nothing to do with the inflow hydrograph; the only link is that the value of Δt used to develop the storage-indication curve must correspond to the time increment of the inflow hydrograph.

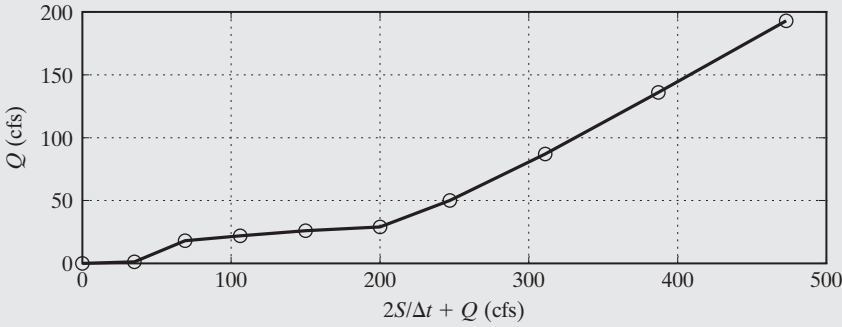


Figure E4-5(c)
Storage-indication
curve.

Step 2 of the procedure is the actual flow routing. Equation (4-13) states:

$$(I_n + I_{n+1}) + \left(\frac{2S_n}{\Delta t} - Q_n \right) = \left(\frac{2S_{n+1}}{\Delta t} + Q_{n+1} \right).$$

For $t_n = 0$, $I_n = 0$, $S_n = 0$, and $Q_n = 0$ (from the known initial conditions). At $t_{n+1} = 10$ min, $I_{n+1} = 20$ cfs, as seen in Figure E4-5(a) and in the flow routing table that follows. Hence,

$$I_n + I_{n+1} = 20 \text{ cfs.}$$

At $t_{n+1} = 10$ min, S_n and $Q_n = 0$ and the left-hand side of Equation (4-13) is $20 + 0 = 20$. Thus, Equation (4-13) becomes

$$20 + (0) = \left(\frac{2S_{n+1}}{\Delta t} + Q_{n+1} \right),$$

$$\left(\frac{2S_{n+1}}{\Delta t} + Q_{n+1} \right) = 20.$$

By linear interpolation within the storage-indication curve, for

$$\left(\frac{2S}{\Delta t} + Q \right) = 20,$$

$$Q = 7.2 \text{ cfs.}$$

For $t_n = 10$ min and $t_{n+1} = 20$ min, $I_n = 20$ cfs and $I_{n+1} = 40$ cfs, hence their sum is 60 cfs. It is not necessary to compute the storage, S_n , to compute the quantity $2S/\Delta t - Q$ at step n . Instead, we subtract twice the flow from the known value of $2S/\Delta t + Q$ at time step n (previous row). Thus,

$$\begin{aligned} \frac{2S_n}{\Delta t} - Q_n &= \left(\frac{2S_n}{\Delta t} + Q_n \right) - 2Q_n \\ &= 20 \text{ cfs} - 2(7.2 \text{ cfs}) \\ &= 5.6 \text{ cfs.} \end{aligned}$$

and

$$I_n + I_{n+1} + \left(\frac{2S_n}{\Delta t} - Q_n \right) = \left(\frac{2S_{n+1}}{\Delta t} + Q_{n+1} \right),$$

$$20 + 40 + 5.6 = 65.6.$$

From the storage-indication curve, for

$$\left(\frac{2S}{\Delta t} + Q \right) = 65.6,$$

$$Q = 17.6 \text{ cfs.}$$

The procedure is shown completed in the accompanying table.

Time (min)	I_{n+1} (cfs)	$(I_n + I_{n+1})$ (cfs)	$\left(\frac{2S_n}{\Delta t} - Q_n \right)$ (cfs)	$\left(\frac{2S_{n+1}}{\Delta t} + Q_{n+1} \right)$ (cfs)	Q_{n+1} (cfs)
0	0	0	0	0	0
10	20	20	0	20	7.2
20	40	60	5.6	65.6	17.6
30	60	100	30.4	130.4	24.0
40	50	110	82.4	192.4	28.1
50	40	90	136.3	226.3	40.4
60	30	70	145.5	215.5	35.5
70	20	50	144.5	194.5	28.2
80	10	30	138.0	168.0	26.6
90	0	10	114.8	124.8	23.5
100	0	0	77.7	77.7	19.0
110	0	0	39.7	39.7	13.5
120	0	0	12.8	12.8	4.6
130	0	0	3.5	3.5	1.3
140	0	0	1.0	1.0	0.4
150	0	0	0.3	0.3	0.1
160	0	0	0.1	0.1	0.0

In the flood routing table, the row for which we are solving for Q is considered to be at step $n + 1$, and the previous row is step n . Note that computations can continue beyond the end of the input hydrograph. If more precise computations are needed, the time step Δt may be shortened and/or the vertical depth increment made smaller that is used to develop the storage indication curve.

The outflow hydrograph is also plotted in Figure E4-5(a). The “bump” at the maximum flow occurs because the water level rises above the weir crest, and the outflow relation changes. To determine the maximum height of the reservoir during this inflow, a depth-vs.-discharge curve may be developed using the data for the storage-indication curve (first table), or the value simply interpolated from the table. The maximum

outflow found in routing is 40.4 cfs. The depth corresponding to this flow, the maximum stage reached by the reservoir, is about 5.2 ft, by linear interpolation in the storage-indication table. If peak outflow is deemed to be too high in this case, then the pond must be enlarged to handle more storage and reduce the peak.

Detention Basin Routing

The purpose of flood routing for detention basin design is to determine how the outflow from a detention basin and the storage in the basin vary with time for a known inflow hydrograph. More accurate numerical routing schemes for solving the continuity and storage equation include the Runge–Kutta methods. Chapra and Canale (2006) discuss **Runge–Kutta** (R-K) methods in detail for solving ordinary differential equations. They can be developed to solve the following equations with various orders of accuracy, where the first-order method is called Euler's method, the second-order method is called Heun's method, and the fourth-order method is referred to as Runge–Kutta.

The continuity equation is expressed

$$\frac{dV}{dt} = Q_{\text{in}}(t) - Q_{\text{out}}(H), \quad (4-14)$$

where

V = volume of water in storage in the basin,
 $Q_{\text{in}}(t)$ = inflow into the detention basin as a function of time,
 $Q_{\text{out}}(H)$ = outflow from a detention basin as a function of head (H) in the basin.

The change in volume dV due to a change in depth dH can be expressed as

$$dV = A_r(H) dH, \quad (4-15)$$

where $A_r(H)$ is the surface area related to H . The continuity equation is then expressed as

$$\frac{dH}{dt} = \frac{Q_{\text{in}}(t) - Q_{\text{out}}(H)}{A_r(H)}. \quad (4-16)$$

Figure 4–5(a) illustrates the first-order method. Equation (4–16) can be represented by

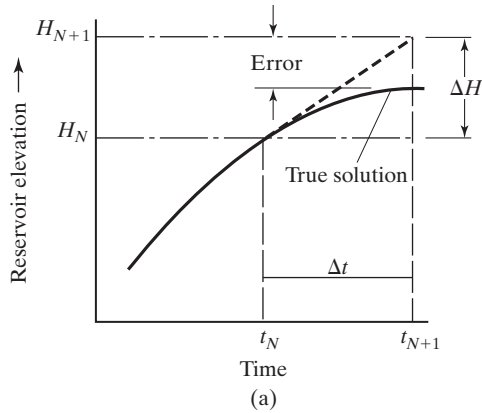
$$\frac{dH}{dt} = f(H_n, t_n),$$

where

H = the dependent variable,
 t = the independent variable.

Figure 4-5a

First-order
Runge-Kutta (Euler)
technique.



In the first-order solution, a finite time increment Δt is chosen. Then

$$\begin{aligned}\Delta H &= f(H_n, t_n) \Delta t, \\ H_{n+1} &= H_n + \Delta H,\end{aligned}\quad (4-17)$$

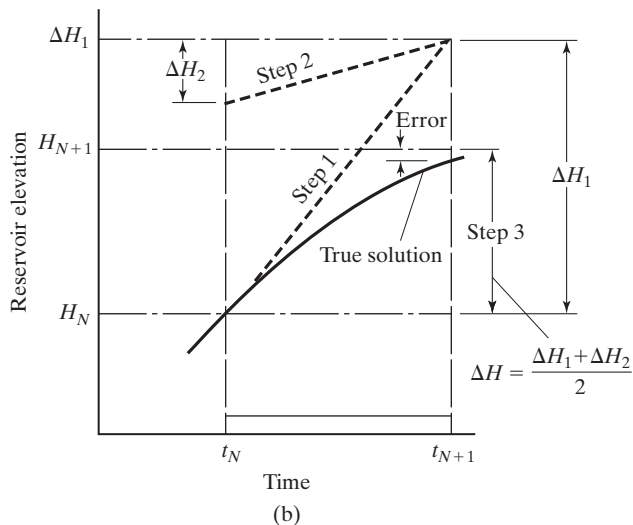
assuming that the initial head H_n is known. However, since ΔH is not constant but is continually changing with time, error is introduced. The second-order R-K technique (Huen method) alleviates some of this error, as shown in Figure 4-5(b), where ΔH is calculated at the beginning and end of the chosen time increment Δt , and the two values of ΔH are averaged.

Note that the line for step 1 is tangent to $f(H_n, t_n)$ at t_n . The line for step 2 is tangent to $f(H_n, t_n)$ at t_{n+1} . First, ΔH_1 is found from

$$\Delta H_1 = f(H_n, t_n) \Delta t.$$

Figure 4-5b

Second-order
Runge-Kutta (Huen)
technique.



Since H_{n+1} has not yet been calculated, ΔH_2 cannot be found at that point. But ΔH_2 is estimated by evaluating Equation (4-17) at $H_n + \Delta H_1$ and $t_n + \Delta t$:

$$\Delta H_2 = f(H_n + \Delta H_1, t_n + \Delta t) \Delta t.$$

Then

$$\begin{aligned} \Delta H &= \frac{\Delta H_1 + \Delta H_2}{2}, \\ H_{n+1} &= H_n + \Delta H. \end{aligned} \quad (4-18)$$

Applying this technique to Equation (4-16), we have

$$\begin{aligned} \Delta H_1 &= \frac{Q_{\text{in}}(t_n) - Q_{\text{out}}(H_n)}{A_r(H_n)} \Delta t, \\ \Delta H_2 &= \frac{Q_{\text{in}}(t_n + \Delta t) - Q_{\text{out}}(H_n + \Delta H_1)}{A_r(H_n + \Delta H_1)} \Delta t. \end{aligned} \quad (4-19)$$

The second-order Huen solution of Equation (4-19) is more accurate than the first-order method.

By reasoning similar to that used in the above solution, the fourth-order R-K solutions attempt to improve the accuracy of the estimate of ΔH . In the classical fourth-order R-K technique,

$$H_{n+1} = H_n + \frac{1}{6}[k_1 + 2k_2 + 2k_3 + k_4] \Delta t, \quad (4-20)$$

where

$$\begin{aligned} k_1 &= f(t_n, H_n), \\ k_2 &= f\left(t_n + \frac{\Delta t}{2}, H_n + \frac{1}{2}k_1\Delta t\right), \\ k_3 &= f\left(t_n + \frac{\Delta t}{2}, H_n + \frac{1}{2}k_2\Delta t\right), \\ k_4 &= f(t_n + \Delta t, H_n + k_3\Delta t). \end{aligned} \quad (4-21)$$

The fourth-order R-K scheme is considered to be the most accurate of the various orders. Homework problems illustrate the use of the R-K solution, and an Excel program can be easily written (see Liengme, 2000).

Hydraulic routing differs from hydrologic routing in that both the equation of continuity and the momentum equation are solved simultaneously rather than through the use of an empirical storage–discharge relation. Since closed-form solutions do not exist for the Saint Venant equations, various numerical methods have been developed for the computer. **Explicit methods** calculate values of velocity and depth over a grid system based on previously

4.4 GOVERNING EQUATIONS FOR HYDRAULIC RIVER ROUTING

known data for the river reach. **Implicit methods** set up a series of simultaneous numerical equations over a grid system for the entire river, and the equations are solved at each time step. **Characteristic methods** employ the concept of characteristic curves in the xt -plane, produced by converting partial differential equations into ordinary differential equations. The equations can be simplified under certain conditions to allow for the use of a uniform flow equation in place of the full momentum equation. This method is referred to as the **kinematic wave model**.

The general equation of continuity states that inflow minus outflow equals rate of change of storage. For the river element shown in Figure 4-6,

$$\begin{aligned}\text{inflow} &= \left(Q - \frac{\partial Q}{\partial x} \frac{\Delta x}{2} \right) \Delta t + q \Delta x \Delta t, \\ \text{outflow} &= \left(Q + \frac{\partial Q}{\partial x} \frac{\Delta x}{2} \right) \Delta t,\end{aligned}\quad (4-22)$$

$$\text{storage change} = \frac{\partial A}{\partial t} \Delta x \Delta t,$$

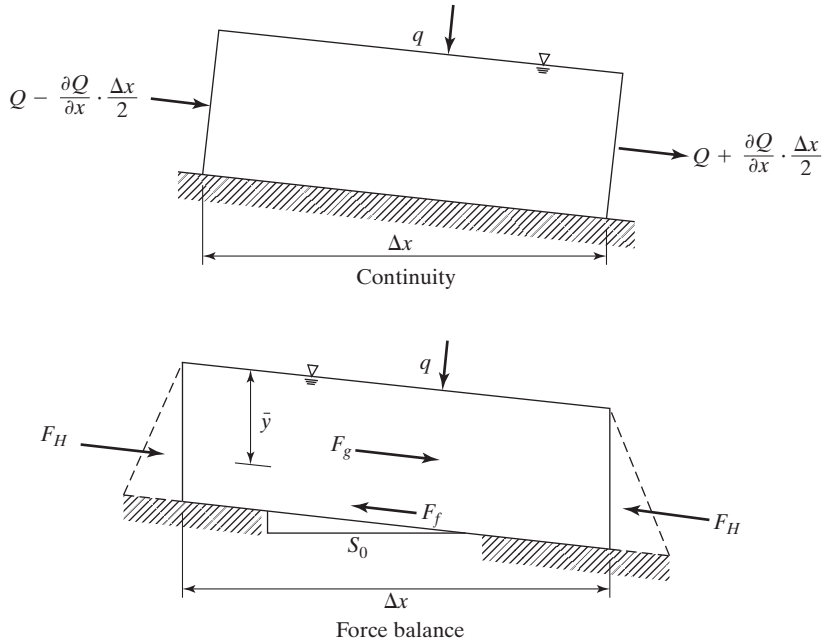
where

q = rate of lateral inflow per unit length of channel,
 A = cross-sectional area.

The equation of continuity becomes, after dividing by Δx and Δt ,

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q. \quad (4-23)$$

Figure 4-6
Continuity and
momentum elements
for a river reach.



For a unit width b of channel with v = average velocity, the continuity equation can be written

$$y \frac{\partial v}{\partial x} + v \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = \frac{q}{b}. \quad (4-24)$$

The momentum equation in the x -direction is produced from a force balance on the river element, according to Newton's second law of motion.

The following three main external forces are acting on area A as shown in Figure 4-6:

$$\text{hydrostatic: } F_H = -\gamma \frac{\partial(\bar{y}A)}{\partial x} \Delta x,$$

$$\text{gravity: } F_G = \gamma A S_0 \Delta x,$$

$$\text{friction: } F_f = -\gamma A S_f \Delta x,$$

where

γ = specific weight of water (ρg),

$\bar{y}$ = distance from the water surface to the centroid of the pressure prism,

S_f = friction slope, obtained by solving for the slope in a uniform flow equation, such as Manning's equation,

S_0 = bed slope.

The hydrostatic force is the net pressure force acting on each end of the reach. The gravity force is along the channel and due to the weight of water in the control volume. The friction force acts along the bottom and sides of the control volume. Other forces that could be included are wind shear and expansion/contraction. The rate of change of momentum is expressed from Newton's second law as

$$F = \frac{d}{dt}(mv),$$

$$\frac{d(mv)}{dt} = m \frac{dv}{dt} + v \frac{dm}{dt} = \rho A \Delta x \frac{dv}{dt} + \rho v q \Delta x, \quad (4-25)$$

where the total derivative of v with respect to t can be expressed

$$\frac{dv}{dt} = \frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x}.$$

Equating Equation (4-25) to the sum of the three external forces above results in

$$\frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + \frac{g}{A} \frac{\partial(\bar{y}A)}{\partial x} + \frac{vq}{A} = g(S_0 - S_f). \quad (4-26)$$

For negligible lateral inflow and a wide channel, the equation can be rearranged to yield (Henderson, 1966)

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{v}{g} \frac{\partial v}{\partial x} - \frac{1}{g} \frac{\partial v}{\partial t}. \quad (4-27)$$

The full dynamic wave equations (Saint Venant equations), presented as Equations (4-23) and (4-27), require numerical techniques for solution and large quantities of measured hydraulic data, including detailed stream cross-section descriptions. These drawbacks can be overcome by simplifying the equations under certain conditions. For overland flow and many channel flow situations, some of the terms in Equation (4-27) can be neglected (Eagleson, 1970). In a typical shallow stream, if the bed slope is 0.01, the rate of change of water depth (dy/dx) will probably not exceed 0.001; the longitudinal velocity gradient term $(v/g)(\partial v/\partial x)$ and the time-rate of change-of-velocity term $(1/g)(\partial v/\partial t)$ will typically be less than 0.001. Thus, the last three terms on the right-hand side of Equation (4-27) can often be neglected. In overland flow conditions, the three terms will be two orders of magnitude less than those for the bed slope. Various flow routing methods will result, depending on the terms neglected (Table 4-1).

The two approaches that have found the widest application in engineering practice are the **kinematic wave** model and the **diffusion model** (Table 4-1).

The kinematic wave model further assumes that the pressure term is negligible, resulting in

$$S_0 = S_f$$

which means that a simple uniform flow formula such as Manning's equation can be used. The diffusion analogy results in the continuity equation [Eq. (4-23)] and the simplified form of the momentum equation [Eq. (4-27)] as

$$\frac{dy}{dx} = S_0 - S_f$$

The diffusion model is described in more detail in Section 4.7. The effect of the diffusion term, compared with the kinematic wave, is to make depth y decrease in the view of an observer moving at velocity c . Thus, attenuation of the flood wave is included in the diffusion model, but not for the kinematic wave. The **dynamic wave** model, which uses the full unsteady nonuniform version of the momentum equation, is only used when both pressure and inertial forces are important.

Table 4-1 Forms of the Momentum Equation

Type of Flow	Momentum Equation
Kinematic wave (steady uniform)	$S_f = S_0$
Diffusion (noninertia) model	$S_f = S_0 - \partial y/\partial x$
Steady nonuniform	$S_f = S_0 - \partial y/\partial x - (v/g)\partial v/\partial x$
Unsteady nonuniform	$S_f = S_0 - \partial y/\partial x - (v/g)\partial v/\partial x - (1/g)\partial v/\partial t$

One of the simplest of all waveforms is the monoclinal flood wave shown in Figure 4-7(a). It is simply a step increase in discharge that moves downstream at wave celerity $u = c$ and with no change in shape. The **kinematic wave** is a stable version of the monoclinal wave and usually occurs over several kilometers so that, to the observer on the bank, flow appears uniform (Ghosh, 1997). Applying the continuity equation (inflow minus outflow equals change in storage) within the reach and neglecting changes in wave-shape results in

$$u(A_2 - A_1) = A_2v_2 - A_1v_1, \quad (4-28)$$

where u and v are velocities of the wave and water, respectively, and A is cross-sectional area of the channel. Thus,

$$u = \frac{A_1v_1 - A_2v_2}{A_1 - A_2} = \frac{Q_1 - Q_2}{A_1 - A_2},$$

where Q is measured relative to the bank. For the wave of small height in a wide rectangular channel, velocity is called the **wave celerity** c and is equal to

$$c = \frac{dQ}{dA} = \frac{1}{B} \frac{dQ}{dy}, \quad (4-29)$$

where B is channel top width. Seddon (1900) first developed Equation (4-29) for the Mississippi River.

The argument leading to Equation (4-29) implies that the wave profile is permanent without any change in shape and that Q is a single-valued function of depth or area. The Manning formula describes average velocity in an open channel:

$$v = \frac{k_m}{n} R^{2/3} \sqrt{S}, \quad (4-30)$$

where

$R = A/P$, the hydraulic radius (ft),

P = wetted perimeter (ft),

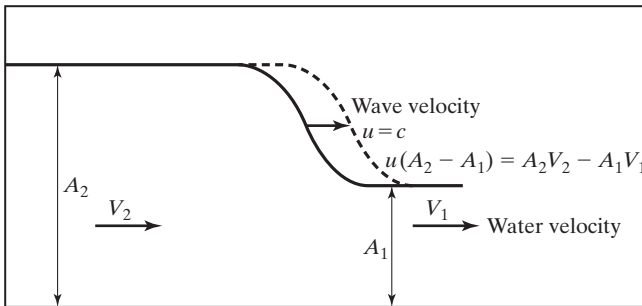


Figure 4-7a
Monoclinal rising
flood wave.

A = cross-sectional area (ft^2),
 S = energy slope (ft/ft),
 n = roughness coefficient (Manning's),
 $k_m = 1.49$ for units of ft and s , and 1.0 for units of m and s .

Then, for a wide rectangular channel where $R \approx y$ and $A = By$, multiplying by area gives the flow rate

$$Q = \frac{k_m}{n} B y^{5/3} \sqrt{S}. \quad (4-31)$$

Differentiating, we have

$$\frac{dQ}{dy} = \left(\frac{5}{3}\right) \frac{k_m}{n} B y^{2/3} \sqrt{S} = \frac{5}{3} B v, \quad (4-32)$$

and substituting into Equation (4-29) gives

$$c = \frac{5}{3} v. \quad (4-33)$$

The ratio of c to v is always greater than unity; that is, wave celerity exceeds velocity for the monoclinal flood wave [Fig. 4-7(b)].

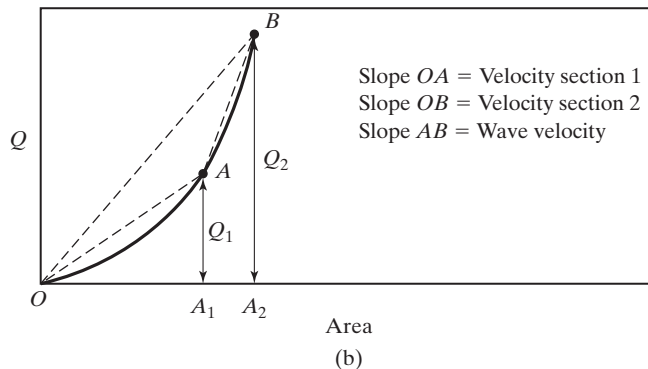
It can be shown (Chow, 1959) that **dynamic waves** (i.e., a gradually varied flood wave that attenuates as it moves downstream) in open channels or tidal bores have wave velocities of

$$c = v \pm \sqrt{gA/B}, \quad (4-34)$$

measured relative to the bank. Thus, an observer on the bank moving at velocity c will see a steady profile (i.e., the wave will appear to not change in time). In Equation 4-34, the cross-sectional area, A , divided by the top width, B , is equal to the hydraulic depth, D . In wide, rectangular channels, the hydraulic depth is equal to the flow depth. This type of dynamic wave has a different velocity than the one presented in Equation (4-28), and it can propagate in either direction. A single-valued relation between Q and

Figure 4-7b

Area–discharge relation for streams.



A is not assumed, and the momentum equation must also be solved for the dynamic wave.

Waves of the form presented in Equation (4-29) are termed **kinematic waves** because they are based on the continuity equation and imply a unique function between Q and y (Lighthill and Whitham, 1955) based on a uniform flow equation. Kinematic waves imply that $S_f = S_0$ and that all other terms in the governing momentum equation [Eq. (4-27)] are negligible.

An alternative derivation of Equation (4-29) comes directly from the continuity equation for a prismatic channel:

$$\frac{\partial Q}{\partial x} + \frac{B \partial y}{\partial t} = 0, \quad (4-35)$$

or

$$\frac{1}{B} \frac{dQ}{dy} \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0, \quad (4-36)$$

but when moving with the wave speed c ,

$$\frac{dy}{dt} = \frac{dx}{dt} \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0. \quad (4-37)$$

Thus,

$$\frac{dx}{dt} = c = \frac{1}{B} \frac{dQ}{dy} \quad (4-38)$$

since Equations (4-36) and (4-37) are both equal to zero. It follows from Equation (4-37) that to an observer moving with velocity c relative to the river bank given by Equation (4-38), y and Q will appear to be constant. Figure 4-8 shows what a stationary observer on the bank sees in the case of dynamic and kinematic waves. Kinematic waves appear as uniform, unsteady flow with water surfaces parallel to the bed and energy grade line.

In a natural flood wave, both kinematic and dynamic waves may be present. The speed of the main flood wave is approximately that of a kinematic wave, with dynamic waves moving ahead of and behind at speeds $v \pm \sqrt{gA/B}$. The speed of the kinematic wave is the same as that of the monoclinal wave, Equation (4-29). If the kinematic wave moves according to Equation (4-33), c will be less than the leading dynamic wave speed, provided that the Froude number is less than 2, where $Fr = v/\sqrt{gA/B}$. This condition occurs in most natural rivers except extremely steep mountain torrents. Given a wave celerity of 10 ft/s and a rate of rise of 5 ft/hr, $\partial y/\partial x = 1/7200$ from Equation (4-37). This implies that only for extremely flat slopes or extremely large rates of rise (as for a dam break) are the kinematic assumptions violated and the water surface slope ($\partial y/\partial x = 1/7200$) becomes important (Ghosh, 1997) (i.e., dynamic waves are present). However, it turns out that, for normal floods in natural rivers, the dynamic wave fronts attenuate very rapidly as long as

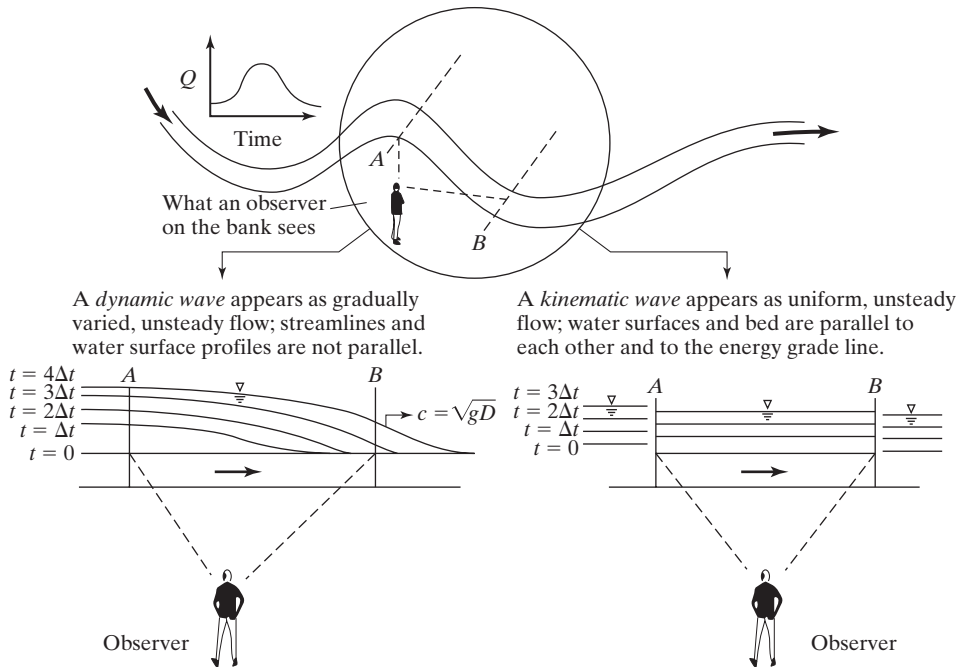


Figure 4-8
Visualization of dynamic and kinematic waves.

$Fr < 2$, and kinematic waves dominate the flood response (Henderson, 1966). More details on kinematic and dynamic flood waves can be found in Chow et al. (1988) and Singh (1996).

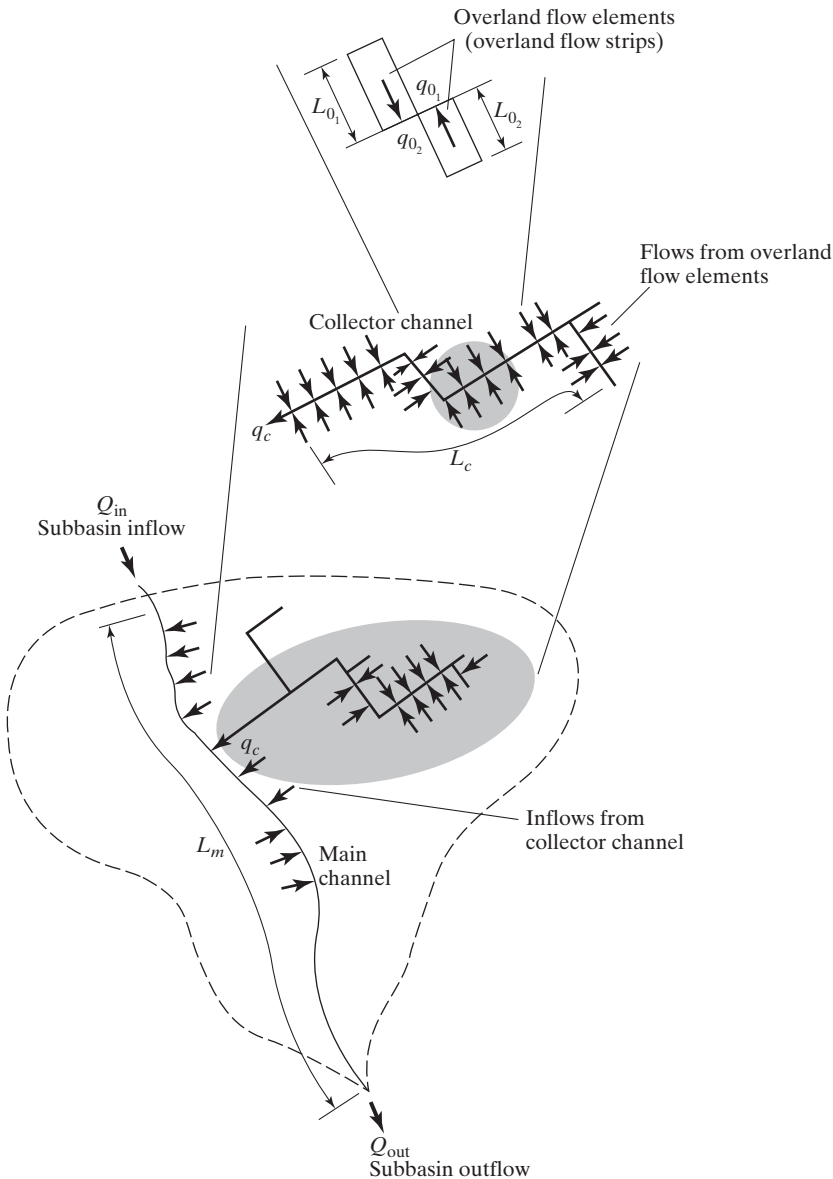
4.6 KINEMATIC WAVE ROUTING The kinematic wave assumption is that inertial and pressure effects are unimportant and that the weight or gravity force of fluid is approximately balanced by the resistive forces of bed friction [Eq. (4-30) and Table 4-1]. Kinematic waves will not accelerate appreciably and can flow only in a downstream direction without any crest subsidence. Hence, kinematic wave methods cannot account for backwater. The flood wave will be observed as a uniform rise and fall in water surface over a relatively long period of time. Thus, kinematic waves represent the characteristic changes in discharge, velocity, and water surface elevation with time at any one location on an overland flow plane or along a stream channel. Kinematic waves are often classified as uniform, unsteady flows.

The kinematic wave method of routing overland and river flows is included in the HEC-HMS program, as described in detail in Chapter 5. The following discussion and numerical solutions are based on the HMS program because of the wide acceptance and availability (Hydrologic Engineering

Center, 2006). Kinematic wave routing is also an important aspect of the Storm Water Management Model (Huber and Dickinson, 1988), as described in Chapter 6, and the Vflo model described in Chapter 11.

The concept incorporated into HEC-HMS uses various elements such as overland flow planes, collector channels, and main channels to route kinematic waves (Fig. 4–9). These various elements are combined to describe basin and subbasin responses to storm events. Overland flow is handled

Figure 4–9
Relationships between flow elements.



separately from open-channel flow because of the assumptions inherent in developing the kinematic flow equation for overland flow planes.

Overland flow in the model is distributed over a wide area and at very shallow average depths until it reaches a well-defined collector channel. Pervious and impervious flow surfaces are allowed in HEC-HMS with unique slopes, flow lengths, roughnesses, and loss rates. After overland runoff is routed down the length of the overland flow strip, it is then routed along the collector system and eventually into a main channel. Runoff moves through the collector system, picking up additional lateral inflow from adjacent strips uniformly distributed along the system. Collector and main channel kinematic wave routing are similar in theory and differ only in the shape of the collector.

Governing Equation for Kinematic Overland Flow Routing

For the conditions of kinematic flow, and with no appreciable backwater effect, the discharge can be described as a function of area only, for all x and t :

$$Q = \alpha A^m, \quad (4-39)$$

where

Q = discharge in cfs,

A = cross-sectional area

α, m = kinematic wave routing parameters.

Henderson (1966) presents the normalized momentum equation [Eq. (4-27)] in the form

$$Q = Q_0 \left(1 - \frac{1}{S_0} \left(\frac{\partial y}{\partial x} + \frac{v}{g} \frac{\partial v}{\partial x} + \frac{1}{g} \frac{\partial v}{\partial t} + \frac{qv}{gy} \right) \right)^{1/2}, \quad (4-40)$$

where Q_0 is the flow under uniform conditions. This equation describes the condition of kinematic flow if the sum of terms to the right of the minus sign is much less than 1. Then

$$Q \approx Q_0. \quad (4-41)$$

It has been shown previously that the kinematic wave form dominates only if $Fr < 2$. Woolhiser and Liggett (1967) analyzed characteristics of the rising overland flow hydrograph and found that the dynamic terms can generally be neglected if

$$k = \frac{S_0 L}{y Fr^2} > 10 \quad \text{or} \quad \frac{S_0 L g}{v^2} > 10, \quad (4-42)$$

where

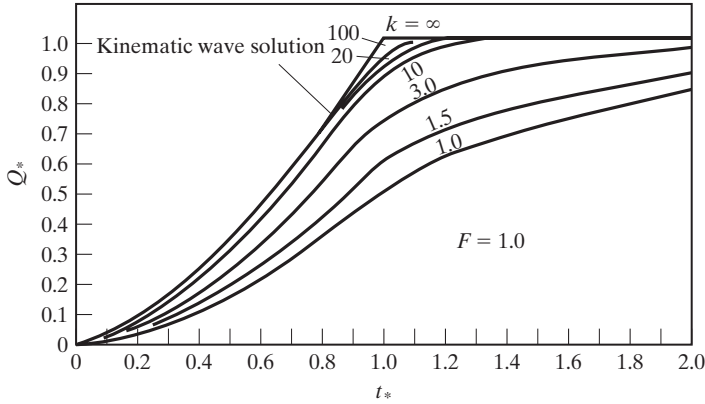
L = the length of the plane,

$Fr = v/\sqrt{gy}$ with v = overland velocity,

y = the depth at the end of the plane,

S_0 = the slope,

k = the dimensionless kinematic flow number.

**Figure 4-10**

Effect of kinematic wave number k on the rising hydrograph. (From Woolhiser and Liggett, 1967.)

These results are best summarized in Figure 4-10, where Q_* (dimensionless flow) is plotted vs. t_* (dimensionless time) for various values of k in Equation (4-42). It can be seen that for $k \leq 10$, large errors in calculated Q_* result by deleting dynamic terms from the momentum equation for overland flow.

The kinematic wave equation for an overland flow segment on a wide plane with shallow flows can be derived from Equation (4-39) and Manning's equation for overland flow:

$$q = \frac{k_m}{n} \sqrt{S_0} y^{5/3}. \quad (4-43)$$

Values of Manning's n for overland flow are typically greater than Manning's n for channels and are presented in Table 4-2, based on field and laboratory investigations.

Rewriting equations in terms of flow per unit width for overland flow q_0 , we have

$$q_0 = \alpha_0 y_0^{m_0}, \quad (4-44)$$

where

$$\alpha_0 = \frac{k_m}{n} \sqrt{S_0} = \text{conveyance factor},$$

$m_0 = 5/3$ from Manning's equation,

S_0 = average overland flow slope,

y_0 = mean depth of overland flow.

The continuity equation is

$$\frac{\partial y_0}{\partial t} + \frac{\partial q_0}{\partial x} = i - f, \quad (4-45)$$

where

i = rate of gross rainfall (ft/s),

f = infiltration rate (ft/s)

q_0 = flow rate per unit width (cfs/ft),

y_0 = mean depth of overland flow (ft).

Table 4-2 Estimates of Manning's Roughness Coefficients for Overland Flow

Source	Ground Cover	n	Range
Crawford and Linsley (1966)*	Smooth asphalt	0.012	
	Asphalt of concrete paving	0.014	
	Packed clay	0.03	
	Light turf	0.20	
	Dense turf	0.35	
Engman (1986)†	Dense shrubbery and forest litter	0.4	
	Concrete or asphalt	0.011	0.01–0.013
	Bare sand	0.01	0.01–0.016
	Graveled surface	0.02	0.012–0.03
	Bare clay-loam (eroded)	0.02	0.012–0.033
	Range (natural)	0.13	0.01–0.32
	Bluegrass sod	0.45	0.39–0.63
	Short-grass prairie	0.15	0.10–0.20
	Bermuda grass	0.41	0.30–0.48

*Obtained by calibration of Stanford Watershed Model.

†Computed by Engman (1986) by kinematic wave and storage analysis of measured rainfall–runoff data.

Finally, by substitution of Equation (4-45) in Equation (4-44), we have

$$\frac{\partial y_0}{\partial t} + \alpha_0 m_0 y_0^{m_0-1} \frac{\partial y_0}{\partial x} = i - f, \quad (4-46)$$

which can be solved numerically for $y_0 = f(x, t, i - f)$. Once y_0 is found, it is substituted into Equation (4-44) to give a value for q_0 . Equations (4-44) and (4-46) form the complete kinematic wave equations for overland flow. Analytical solutions for runoff from an impermeable plane surface are presented in more advanced texts (Singh, 1996), and will not be presented here. Numerical techniques for solving these equations are described in more detail in the section on finite differences.

Kinematic Channel Routing

Simple cross-sectional shapes such as triangles, trapezoids, and circles are used as representative collectors or stream channels. These are completely characterized by slope, length, cross-sectional dimensions, shape, and Manning's n value. The basic forms of the equations are similar to the overland flow Equations (4-44) and (4-45). For stream channels or collectors,

$$\frac{\partial A_c}{\partial t} + \frac{\partial Q_c}{\partial x} = q_0, \quad (4-47)$$

$$Q_c = \alpha_c A_c^{m_c}, \quad (4-48)$$

where

- A_c = cross-sectional flow area (ft^2),
- Q_c = discharge (cfs),
- q_0 = overland inflow per unit length (cfs/ft),
- α_c, m_c = kinematic wave parameters for the particular channel.

The values of α_c and m_c are derived for a simple triangular section in Example 4-6, and results are then presented only for rectangular, trapezoidal, and circular shapes in Table 4-3. Figure 4-11 presents shape parameters for typical channels.

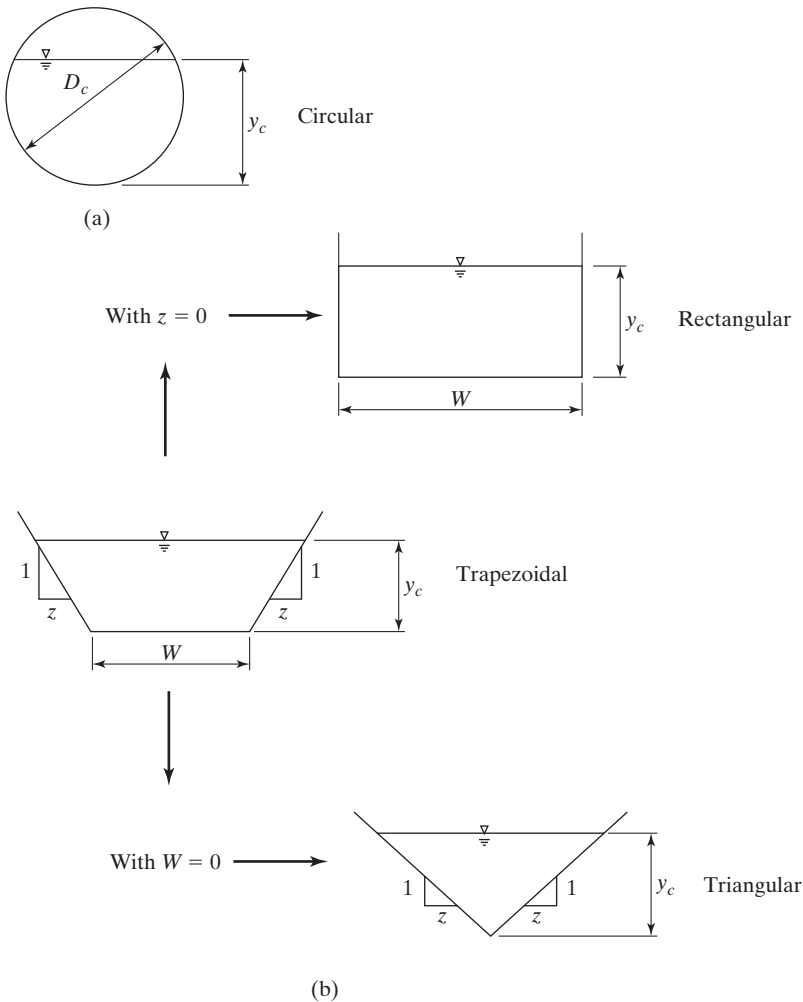


Figure 4-11

Basic channel shapes and their variations used by the HEC-HMS flood hydrograph package for kinematic wave stream routing.

Table 4-3 Kinematic Channel Parameters*

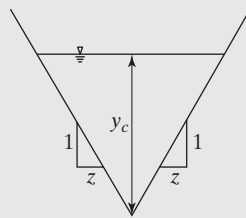
Shape	U.S. Customary Units	
	α_c	m_c
Square	$\frac{0.72 \sqrt{S}}{n}$	4/3
Rectangle	$\frac{1.49 \sqrt{S}}{n} (W^{-2/3})$	5/3
Trapezoidal	Variable, function of A and W	
Circular	$\frac{0.804 \sqrt{S}}{n} (D_c^{1/6})$	5/4

*From Hydrologic Engineering Center, 1981.

EXAMPLE 4-6**KINEMATIC CHANNEL PARAMETERS**Determine α_c and m_c for the case of a triangular prismatic channel.**SOLUTION**

The geometry of the channel cross section is shown in Figure E4-6.

$$\begin{aligned} \text{area} &= A_c = zy_c^2 \text{ and } y_c = \text{channel depth,} \\ \text{wetted perimeter} &= P_c = 2y_c \sqrt{1 + z^2}, \\ \text{hydraulic radius} &= R = A_c / P_c. \end{aligned}$$

Figure E4-6

Substituting these into Manning's equation for U.S. Customary Units [Eq. (4-30)], we have

$$\begin{aligned} Q_c &= \frac{1.49}{n} \sqrt{S} \frac{A_c^{5/3}}{P_c^{2/3}} \\ &= \frac{1.49 \sqrt{S}}{n} \frac{(z^{5/3} y_c^{10/3})}{1.59 y_c^{2/3} (1 + z^2)^{1/3}} \\ &= \frac{0.94 \sqrt{S}}{n} \left(\frac{z}{1 + z^2} \right)^{1/3} (zy_c^2)^{4/3} \\ &= \frac{0.94 \sqrt{S}}{n} \left(\frac{z}{1 + z^2} \right)^{1/3} A_c^{4/3}. \end{aligned}$$

From Equation (4-48), $Q_c = \alpha_c A_c^{m_c}$. Therefore,

$$\alpha_c = \frac{0.94\sqrt{S}}{n} \left(\frac{z}{1+z^2} \right)^{1/3},$$

$$m_c = 4/3.$$

Finite-Difference Approximations

Finite-difference approximations must be used to solve the kinematic wave equations for actual watersheds and variable rainfalls and can be derived by considering a continuous function $y(x)$ and its derivatives (Chapra and Canale, 2006). A Taylor series expansion of $y(x)$ at $x + h$, where $h = \Delta x$, is

$$y(x + h) = y(x) + hy'(x) + \frac{h^2}{2!}y''(x) + \frac{h^3}{3!}y'''(x) + \cdots, \quad (4-49)$$

and the Taylor series expansion at $x - h$ is

$$y(x - h) = y(x) - hy'(x) + \frac{h^2}{2!}y''(x) - \frac{h^3}{3!}y'''(x) + \cdots, \quad (4-50)$$

where $y'(x)$, $y''(x)$, and $y'''(x)$ are the first, second, and third derivatives of $y(x)$, respectively.

A central-difference approximation is defined by subtracting the expansion Equations (4-49) and (4-50),

$$y(x + h) - y(x - h) = 2hy'(x) - \frac{2h^3}{3!}y'''(x) + \cdots, \quad (4-51)$$

and then solving for $y'(x)$, assuming the third- and higher-order forms are negligible ($|y'''(x)| \ll 1$), so that

$$y'(x) = \frac{y(x + h) - y(x - h)}{2h}, \quad (4-52)$$

which has an error of approximation (truncation error) of order h^2 .

A forward difference is defined by considering the expansion, Equation (4-49), as

$$y(x + h) = y(x) + hy'(x) + \cdots. \quad (4-53)$$

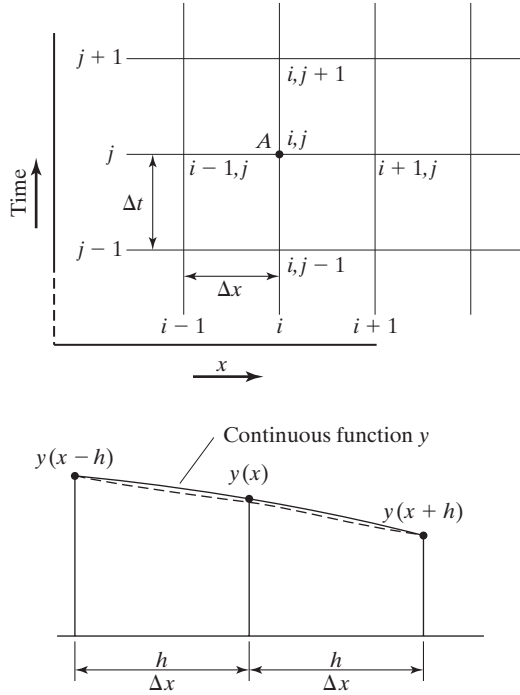
Solving for $y'(x)$ and assuming the second- and higher-order terms are negligible, we have

$$y'(x) = \frac{y(x + h) - y(x)}{h}, \quad (4-54)$$

which has an error of approximation of order h .

Figure 4-12

Illustration of finite-difference grid and method.



The backward-difference approximation can be determined considering the expansion, Equation (4-50), and solving for $y'(x)$:

$$y'(x) = \frac{y(x) - y(x - h)}{h}. \quad (4-55)$$

An xt -plane (Fig. 4-12) consisting of a rectangular net of discrete points is used to represent the continuous solution domain defined by the independent variables (x and t) in the governing flow equations [Eqs. (4-44) and (4-46)]. The lines parallel to the x -axis are spaced by the time increment. The discrete points in the solution domain are defined by the subscript i for the x -position and the subscript j for the time line. More details on finite-difference approximations can be found in Chapra and Canale (2006).

Explicit Solution for Kinematic Wave

The governing kinematic wave equation to be solved is of the form

$$\frac{\partial A_c}{\partial t} + \alpha_c m_c A_c^{m_c-1} \frac{\partial A_c}{\partial x} = q_0, \quad (4-56)$$

where Equation (4-48) has been substituted into Equation (4-47). These methods are included into the HEC-HMS Flood Hydrograph package

(Hydrologic Engineering Center, 1981, 1990), and coding information and boundary conditions are presented there. Further details on numerical methods for kinematic wave routing are contained in Overton and Meadows (1976), Stephenson and Meadows (1986), Bras (1990), and Singh (1996).

The governing equations are similar in form for overland flow, collector elements, and main channels; therefore, solution details are presented only for the collector elements, since the others are basically the same. Finite-difference approximations are made for the various derivatives in Equation (4-56) over a grid space in x and t . Explicit computations advance along the x -dimension downstream for each time step Δt until all the flows and stages are calculated along the entire distance L . The time increment is advanced by Δt , and the procedure is repeated once again. Figure 4-12 shows the grid space in x and t over which the solution is found.

The standard form of the equation solves for the values of A_c and Q at point A in Figure 4-12 as a function of known values at previous points in x and t . Thus, using backward differences and an explicit solution results in

$$\begin{aligned} \frac{A_{ij} - A_{ij-1}}{\Delta t} + \alpha_c m_c \left(\frac{A_{ij-1} + A_{i-1,j-1}}{2} \right)^{m_c-1} \times \left(\frac{A_{ij-1} + A_{i-1,j-1}}{\Delta x} \right) \\ = \frac{q_{ij} + q_{ij-1}}{2}, \end{aligned} \quad (4-57)$$

where two of the terms are averaged over two points in space or time. We solve for A_{ij} (directly as a function of other known values) and then compute Q_{ij} from

$$Q_{ij} = \alpha_c (A_{ij})^{m_c}. \quad (4-58)$$

We can also compute y for a given flow, time, and location, knowing Q and A .

This method requires the Courant condition to be met, that is, $c \Delta t / \Delta x \leq 1$ to maintain numerical stability (Hydrologic Engineering Center, 1981), where c is the kinematic wave celerity defined earlier in Equation (4-29). However, if the Courant condition is violated, or $c > \Delta x / \Delta t$, the numerical scheme must be modified to maintain numerical stability. The conservative form of Equation (4-47) solves first for Q_{ij} as

$$Q_{ij} = Q_{i-1,j} + \bar{q} \Delta x - [\Delta x / \Delta t] [A_{i-1,j} - A_{i-1,j-1}],$$

where

$$\bar{q} = \frac{q_{ij} + q_{ij-1}}{2}. \quad (4-59)$$

Knowing Q_{ij} , we find A_{ij} by solving

$$A_{ij} = \left(\frac{Q_{ij}}{\alpha_c} \right)^{1/m_c}. \quad (4-60)$$

Some numerical instability can occur under certain conditions using the above explicit method, since c may vary in space and time. Chow et al. (1988) present an alternative scheme that solves for Q as the only dependent variable by combining the continuity and momentum equations together. They solve the following governing equation using a finite-difference scheme similar to the one above,

$$\frac{\partial Q}{\partial x} + \left[\frac{1}{\alpha} \right]^{1/m} \frac{1}{m} Q^{1/m-1} \frac{\partial Q}{\partial t} = q, \quad (4-61)$$

where

$$A = \left[\frac{Q}{a} \right]^{1/m}$$

for the momentum equation.

4.7 HYDRAULIC RIVER ROUTING

The application of the Saint Venant equations [Eqs. (4-23) and (4-27)] in flood routing involves solving them simultaneously down the length of a stream channel. An excellent review of methods is given by Price (1974), who compares four of the more important numerical methods for hydraulic flood routing with exact analytical solutions for the monoclinal wave (Section 4.5). Price found that the four-point implicit numerical scheme of Amein and Fang (1970) was the most efficient of the methods and is most accurate when $\Delta x/\Delta t$ is approximately equal to the speed of the monoclinal wave, the kinematic wave speed. In the numerical schemes, the stage was defined at the upstream boundary, and rating curve conditions were specified at the downstream boundary.

The objective of the numerical flood routing method is to simulate the propagation of a flood downstream in a river, given a stage or discharge hydrograph at an upstream section. The preceding methods provide the most accurate solution available for predicting floods downstream in a large river or for considering effects of channel alteration or reservoir storage. Because the computational effort is relatively large for any numerical solution, careful consideration should be given to selecting one of the available hydraulic methods vs. one of the simpler, approximate methods such as kinematic wave or the Muskingum method.

Characteristic Methods

The early approaches to numerical flood routing were based on the characteristic form of the governing equations. This form basically replaces the partial differential equations with four ordinary differential equations, which are then solved numerically along the characteristic curves. Henderson (1966) and Overton and Meadows (1976) present details of the conversion. However, the characteristic method is cumbersome because of required interpolation over

the variable grid mesh in space and time. The method involves integration along two sets of characteristic curves, and a network of variable points is located in the xt -plane by the intersection of the forward and backward characteristic curves. With the advent of the more efficient implicit methods described in the sections that follow, the characteristic method offers no additional accuracy with relatively complex interpolation problems.

Explicit Methods

Explicit methods are primarily the outcome of the pioneering work of Stoker (1957) and Issacson et al. (1956). A network of nodes, as shown in Figure 4-13, is defined for solving the governing equations in x and t . The variables are known at points L , M , and R . The centered difference solution is used to solve Equations (4-23) and (4-27) for velocity $v(P)$ and depth $y(P)$ at point P . The initial condition and boundary conditions (upstream and downstream) must be known from inflow hydrographs and stage-outflow relations. The major disadvantage of the explicit approach is the requirement of using small time steps, known as the Courant condition, to avoid stability problems [$\Delta t \leq \Delta x/c$, where c is defined in Eq. (4-34)].

Implicit Methods

Implicit methods have received the most interest in recent years because they overcome the stability problems of explicit methods and allow fewer time steps for flood routing. The four-point implicit method of Amein and Fang (1970) has the feature of solving the nonlinear simultaneous finite-difference equations using the Newton iteration technique. In this method, a set of N equations with N unknowns results from writing the equations in space and time and incorporating upstream and downstream boundary conditions. The details of the iteration process are given in Amein and Fang (1970) with relevant boundary conditions. The method was applied to several actual flood predictions in North Carolina and was found to be accurate and more efficient than the explicit or characteristic methods. Computer

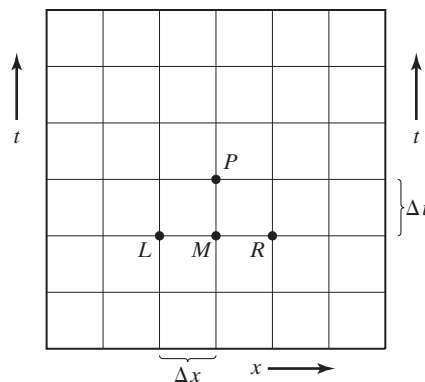


Figure 4-13
Grid used for explicit
solution.

time can be further reduced using the Gaussian elimination procedure suggested by Fread (1971). Comparisons by Price (1974) with exact analytical solutions for the monoclinal wave also found the four-point implicit method to be most efficient of the ones tested. More details can be found in Ponce (1989) and Chow et al. (1988). Numerical methods are presented in detail in Chapra and Canale (2006).

Hydraulic variables at three nodes are known from initial and boundary conditions, and all unknowns are evaluated at a fourth point at the next time step. The resulting set of algebraic finite-difference equations is nonlinear and must be solved using an iterative scheme. The solution procedure solves for all unknowns at one advanced time level simultaneously, and larger Δx and Δt values can be used than in the explicit method.

Diffusion Model

The diffusion wave model (also called the noninertia model) is intermediate in complexity between the kinematic wave model and the full dynamic wave model and assumes that the inertia terms in Equation (4-27) are negligible compared with the pressure, friction, and gravity terms. The momentum equation becomes

$$\frac{\partial y}{\partial x} = S_0 - S_f, \quad (4-62)$$

and when combined with the continuity Equation (4-23), the resulting equation resembles a form of the classic diffusion equation in mathematics:

$$\frac{\partial Q}{\partial t} + c \frac{\partial Q}{\partial x} = D_1 \frac{\partial^2 Q}{\partial x^2}, \quad (4-63)$$

where c is wave celerity and D_1 is a diffusion coefficient (defined in the next section). The effect of the diffusion term, compared with the kinematic wave, is to make y decrease in the view of an observer moving at velocity c . Thus, attenuation of the flood wave is included in the diffusion model, but not for the kinematic wave. Attenuation may be important in long rivers or for rivers with large floodplains.

The kinematic model applies to shallow flow on steep slopes or to long-duration flood waves. The diffusion model applies to a wider range of conditions, and the full dynamic model applies to any condition.

Muskingum–Cunge Method

The Muskingum method presented in Section 4.2 is based on the continuity equation and the storage–discharge relation. Cunge (1969) extended the method into a finite-difference scheme called the Muskingum–Cunge method that takes the form

$$\frac{dS}{dt} = K \frac{d}{dt} [xQ_j + (1-x)Q_{j+1}] = Q_j - Q_{j+1}, \quad (4-64)$$

where

$$\begin{aligned} Q_j &= \text{inflow to reach,} \\ Q_j + 1 &= \text{outflow from reach,} \\ S &= \text{storage in the reach,} \\ K &= \text{travel-time parameter,} \\ x &= \text{weighting factor } (0 \leq x \leq 0.5). \end{aligned}$$

In finite-difference form,

$$\begin{aligned} &\frac{K}{\Delta t} [xQ_j^{n+1} + (1-x)Q_{j+1}^{n+1} - xQ_j^n - (1-x)Q_{j+1}^n] \\ &= \frac{1}{2} (Q_j^{n+1} - Q_{j+1}^{n+1} + Q_j^n - Q_{j+1}^n). \end{aligned} \quad (4-65)$$

Cunge then showed that if $K = \Delta x/c$, then Equation (4-65) is the finite-difference form of the original kinematic wave equation

$$\frac{\partial Q}{\partial t} + c \frac{\partial Q}{\partial x} = 0. \quad (4-66)$$

Cunge then showed that if $D_1 = (1/2 - x)c \Delta x$, then the diffusion equation results and has the advantage of including attenuation in the flood wave based on stream characteristics. The diffusion equation is

$$\frac{\partial Q}{\partial t} + c \frac{\partial Q}{\partial x} = D_1 \frac{\partial^2 Q}{\partial x^2}. \quad (4-67)$$

If the diffusion coefficient is defined as

$$D_1 = \frac{Q_p}{2 B S_0},$$

then

$$x = \frac{1}{2} - \frac{D_1}{c D x}, \quad (4-68)$$

where B is the top width, Δx is the subreach length, Q_p is the peak flow, and S_0 is the channel slope.

According to the Cunge method, the outflow hydrograph at the downstream end is calculated similarly to Equation (4-6) for the Muskingum method, using

$$Q_{j+1}^{n+1} = C_1 Q_j^n + C_2 Q_j^{n+1} + C_3 Q_{j+1}^n + C_4, \quad (4-69)$$

where

$$C_1 = \frac{Kx + \Delta t/2}{D}, \quad (4-70)$$

$$C_2 = \frac{\Delta t/2 - Kx}{D}$$

$$C_3 = \frac{K(1 - x) - \Delta t/2}{D}, \quad (4-71)$$

$$C_4 = \frac{q\Delta t\Delta x}{D},$$

$$D = K(1 - x) + \frac{\Delta t}{2}.$$

Thus, computationally, the Muskingum and Muskingum–Cunge methods are identical. The advantage of the latter is as an aid in parameter selection ($K, x, \Delta t, \Delta x$) based on stream characteristics. Cunge (1969) showed for numerical stability it is required that $0 < x < 1/2$. A complete example of the Muskingum–Cunge routing procedure is presented in Example 4–7, as adapted from Raudkivi (1979).

EXAMPLE 4–7

MUSKINGUM–CUNGE METHOD

The hydrograph at the upstream end of a river is given in the following table. The reach of interest is 18 km long. Using a subreach length Δx of 6 km, determine the hydrograph at the end of the reach using the Muskingum–Cunge method. Assume $c = 2$ m/s, $B = 25.3$ m, $S_0 = 0.001$, and no lateral inflow.

Time (hr)	Flow (m ³ /s)
0	10
1	12
2	18
3	28.5
4	50
5	78
6	107
7	134.5
8	147
9	150
10	146
11	129
12	105
13	78
14	59
15	45
16	33
17	24
18	17
19	12
20	10

Set

SOLUTION

$$K = \Delta x / c$$

$$= \frac{6 \text{ km} \cdot 1000 \text{ m/km}}{2 \text{ m/s}}$$

$$K = 3000 \text{ sec.}$$

Then Equation (4-68) gives

$$x = \frac{1}{2} - \frac{Q_p}{2 B S_{0c} \Delta x}, \quad \text{where } Q_p = 150 \text{ m}^3/\text{s}$$

$$= \frac{1}{2} - \frac{150 \text{ m}^3/\text{s}}{2(2)(25.3)(.001)(6000) \text{ m}^3/\text{s}}$$

$$x = 0.253.$$

No lateral inflow means $q = 0$. Δt is found from Figure 4-14 for $x = 0.253$:

$$\Delta x / (c \Delta t) \leq 0.82,$$

$$\Delta t > \Delta x / (c)(0.82),$$

$$\Delta t > \frac{6 \text{ km} \cdot 1000 \text{ m/km}}{2 \text{ m/s} \cdot 0.82}$$

$$\Delta t > 3658 \text{ s.}$$

Use

$$\Delta t = 7200 \text{ sec (120 min).}$$

Then Equations (4-70) and (4-71) give

$$D = K(1 - x) + \frac{1}{2} \Delta t$$

$$= 50(1 - 0.253) + 0.5(120)$$

$$D = 97.33 \text{ min,}$$

$$C_1 = \frac{Kx + \Delta t/2}{D}$$

$$= [50(0.253) + 0.5(120)]/97.33$$

$$C_1 = 0.7466,$$

$$C_2 = 0.4863,$$

$$C_3 = -0.2329,$$

$$C_4 = \frac{q \Delta t \Delta x}{D} = 0.$$

Check

$$\Sigma C_i = 0.7466 + 0.4863 - 0.2329 = 1.0.$$

Then

$$Q_{j+1}^{n+1} = C_1 Q_j^n + C_2 Q_j^{n+1} + C_3 Q_{j+1}^n + C_4 \quad (4-69)$$

For $j = 0$, $\Delta x = 0$, and $\Delta t = 2$ hr. Thus, the hydrograph for $j = 0$ is given in column 3 of the following table. For any j and $n = 0$ (time 0), the flow is assumed to be $10 \text{ m}^3/\text{s}$. For $j = 0$, $n = 0$:

$$\begin{aligned} Q_1^1 &= C_1 Q_0^0 + C_2 Q_0^1 + C_3 Q_1^0, \\ &= (0.7466)(10) + (0.4863)(18) + (-0.2329)(10), \\ Q_1^1 &= 13.89 \text{ m}^3/\text{s}. \end{aligned}$$

For $j = 0$, $n = 1$,

$$\begin{aligned} Q_1^2 &= C_1 Q_0^1 + C_2 Q_0^2 + C_3 Q_1^1 \\ &= (0.7466)(18) + (0.4863)(50) + (0.2329)(13.89), \\ Q_1^2 &= 34.51 \text{ m}^3/\text{s}. \end{aligned}$$

The process is continued through $j = 3$, $n = 14$, to obtain the following values, where $\Delta t = 2$ hr and $\Delta x = 6$ km.

$n \Delta t$ (hr)	$j \Delta x$ (km)			
	0	6	12	18
0	10	10	10	10
2	18	13.89	11.89	10.92
4	50	34.51	24.38	18.19
6	107	81.32	59.63	42.96
8	147	132.44	111.23	88.60
10	146	149.91	145.88	133.35
12	105	125.16	138.82	145.37
14	59	77.93	99.01	117.94
16	33	41.94	55.52	73.45
18	17	23.14	29.63	38.75
20	10	12.17	16.29	21.02
22	10	9.49	9.91	12.09
24	10	10.12	9.70	9.30
26	10	9.97	10.15	10.01
28	10	10.01	9.95	10.08

Rating Curves

Rating curves are plots of water level (stage) vs. discharge. These are developed from continuous records of water level coupled with discrete discharge measurements made at times of various river stages to produce a unique relationship between the two for a particular location and cross section. However, unless flow can be approximated as uniform ($S_f = S_0$), discharge

will be greater during the rising stage of a flood hydrograph than during the falling stage, leading to a loop rating curve, as shown in Figure 4-14. If the hydrograph has multiple peaks, the curve can be very complex, as shown in Figure 4-14(a). A theoretical shape for a smooth, single-peak hydrograph is shown in Figure 4-14(b).

The physical explanation for the hysteresis present in the rating curve lies primarily with the water surface slope. It is the next most significant term in the momentum equation [Eq. (4-27)]; if it is retained, it leads to diffusion or noninertial routing [Eq. (4-62)]. When the momentum equation is rearranged into the form of Equation (4-40), it is seen that the flow

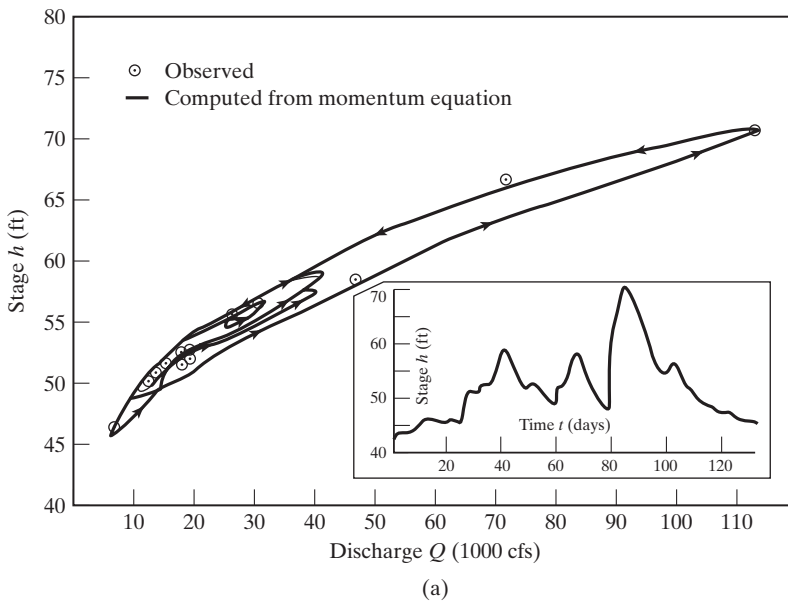


Figure 4-14a

Loop rating curves. Stage-discharge relation for Red River, Alexandria, Louisiana. (From Fread, 1978.)

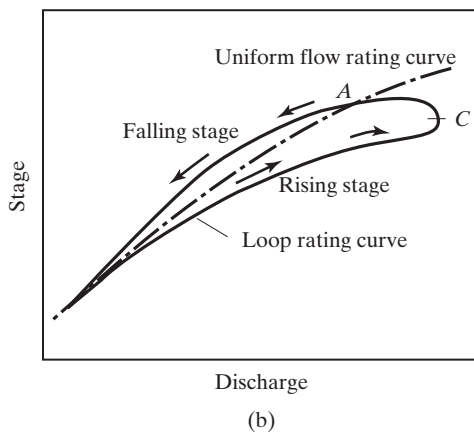


Figure 4-14b

Loop rating curves. Theoretical rating curve. (Henderson, F. M., *Open Channel Flow*, 1st ed., © 1966. Reprinted and Electronically reproduced by permission of Pearson Education, Inc., Upper Saddle River, New Jersey.)

equals uniform flow multiplied by a factor that depends on the water surface slope, a lateral inflow term (the last term), and two inertial terms of lesser significance. During the rising portion of the flood wave, the water surface slope is in the same direction as the slope of the bed S_0 and acts to accelerate the flow [the slope appears as wedge storage in Fig. 4-3(a)]. During this time, the slope $\partial y/\partial x$ is negative and adds to the flow in Equation (4-40). After the flood crest has passed, the water surface slope is positive [Fig. 4-3(c)] and acts to decrease the flow in Equation (4-40). Physically, the positive water surface slope acts to move water in the reverse direction, thus decelerating the positive flow.

The water surface slope effect can be approximated using depth vs. time data in the following manner. Retaining only the slope term in Equation (4-40) gives

$$\frac{Q}{Q_0} = \sqrt{1 - \frac{1}{S_0} \frac{\partial y}{\partial x}}. \quad (4-72)$$

The kinematic wave equation (Eq. 4-37) can be used to approximate the slope as

$$\frac{\partial y}{\partial x} = -\frac{1}{c} \frac{\partial y}{\partial t}, \quad (4-73)$$

where c = kinematic wave celerity. Substituting Equation (4-73) into Equation (4-72) leads to the Jones formula (Henderson, 1966):

$$\frac{Q}{Q_0} = \sqrt{1 + \frac{1}{S_0 c} \frac{\partial y}{\partial t}}. \quad (4-74)$$

Here it can be seen that flow is greater than uniform during periods of rising stage ($\partial y/\partial t > 0$) and less than uniform during periods of falling stage, leading to hysteresis and the loop rating curve. The advantage of the Jones formula is that it involves only terms that can be measured at a single gaging location, whereas computation of $\partial y/\partial x$ must involve at least two locations. Additional corrections to Equation (4-74) can be made to account for subsidence of the flood wave (Henderson, 1966; Stephenson and Meadows, 1986).

Interesting features of the loop rating curve are described by Henderson (1966). Point *C* in Figure 4-14(b) is the point of maximum flow, and point *A* is the point of maximum stage. Thus, an observer on the bank watching a flood will experience the maximum flow rate before experiencing the maximum stage. The kinematic wave method uses the uniform flow relationship shown in Figure 4-14(b) and thus cannot simulate the loop rating curve.

The USGS computes rating curves for all of its gaging stations; these are usually plotted on logarithmic scales, which tend to "straighten" the fit (Kennedy, 1983). Fortunately, most gaging data tend to produce rating curves that can be represented as single valued, thus avoiding the complex

problems of hysteresis just discussed. The USGS website listed in Appendix E contains data on rating curves at selected sites.

Unsteady or Dynamic Models

Hydrologic routing techniques are not adequate for modeling unsteady flows subject to backwater or tidal effects and channels with very mild bottom slopes. Chapter 7 discusses the unsteady state version of HEC-RAS which uses UNET as the internal program for computations. The St. Venant equations, as described earlier, are typically used to solve unsteady flow problems. As a result of the development of computer hardware and software technology, simulation software became widely available to solve the full St. Venant equations in the 1980s and 1990s. In the United States, there are two widely used dynamic models using the St. Venant equations, FLDWAV and UNET. The FLDWAV model that evolved from its earlier versions DAM-BRK (1984) and DWOPER (1978) was developed by D. Fread and has been mainly applied to the national river forecast; while the UNET model was developed by Barkau and has been incorporated into the USACE Hydrologic Engineering Center's River Analysis Systems (HEC-RAS) in order to simulate one-dimensional unsteady flow through a full network of open channels (Barkau, 1992; HEC, 2010).

Discrete solution over a grid and iteration are the two major processes in the computational process in solving the St. Venant Equations. HEC-RAS uses an implicit finite-difference scheme in the stage of discretization to determine the new values of water level and discharge at a discrete point based on all discrete points and with at least two time steps solved simultaneously. This implicit scheme is found more stable than other explicit schemes. Both HEC-RAS and DWOPER use the four-point numerical scheme. Solving the implicit equations requires an iteration approach, which means that at a given time step, successive solutions of water level and discharge are obtained until the difference of the latest values and the values from the previous iteration is small enough. Thus, it is regarded to have converged on the correct value. For instance, HEC-RAS uses 0.02 ft and 20 as the default water-level convergence and the maximum number of iterations.

The unsteady flow component in HEC-RAS was developed primarily for subcritical flow regime calculations. But with version (4.1) of HEC-RAS, the model is able to perform mixed flow regime (subcritical, super-critical, hydraulic jumps, and draw downs) calculations in the unsteady flow computation module. The hydraulic calculations for cross sections, bridges, culverts, and other hydraulic structures that were developed for the steady flow component were incorporated into the unsteady flow module. Additionally, the unsteady flow component is capable of handling dam break analysis, levee breaching and overtopping, pumping stations, navigation dam operations, and pressurized pipe systems. Further details of the unsteady flow component can be found in the manual provided with HEC-RAS (HEC, 2010).

SUMMARY Chapter 4 covers hydrologic (storage) and hydraulic routing methods to predict flood movement for rivers and reservoirs. Hydrologic routing requires solution of the lumped continuity equation and a storage–outflow relation. The relation may be a function of both inflow and outflow for a river, but for a reservoir, storage is related only to outflow. Simple numerical methods are presented for solving the flood routing equations through time.

Hydraulic river routing is more complex than hydrologic routing in that both the one-dimensional continuity and the momentum equations must be solved. In many cases, the momentum equation may be simplified to Manning’s equation; the procedure is called kinematic wave routing. Most practical kinematic wave solutions require numerical methods to compute actual hydrographs, such as those that are currently included in the HEC-HMS (HMS) Flood Hydrograph package or the Vflo model (Vieux, 2004).

Full hydraulic (dynamic) routing requires solution of the Saint Venant equations, for which advanced numerical schemes must be employed to maintain mathematical stability and provide accurate solutions. Implicit finite-difference methods are the most commonly used techniques. One of the most accurate computer models for dynamic routing is DWOPER, which uses the four-point implicit method. DWOPER (as well as the newer NWS FLDWAV) has been extensively tested for floods on the Mississippi, Ohio, and Columbia Rivers with excellent results.

The UNET model was developed by Barkau (1992) and has been incorporated into the USACE HEC-RAS model in order to simulate one-dimensional unsteady flow through a full network of open channels (HEC, 2010). This model is currently widely used for applications to flood evaluation and analysis at all scale levels.

**CONCEPT
CHECK**

- 4.1. How does river routing differ from reservoir routing?
- 4.2. Compare and contrast hydrologic and hydraulic routing methods.
- 4.3. What are the assumptions made for Kinematic Wave Routing? As a result of these assumptions, what are the shortcomings of the method?
- 4.4. How does the method in Problem 4.20 compare with the kinematic wave routing method described in this chapter? What hydraulic conditions are necessary for kinematic wave assumptions to be valid for overland flow? What are advantages and disadvantages of using kinematic channel routing compared with solving the Saint Venant equations?
- 4.5. What is a rating curve? How does a loop rating curve occur?

PROBLEMS

- 4.6. An inflow hydrograph is measured for a cross section of a stream. Compute the outflow hydrograph at a point 5 miles downstream using the Muskingum method. Assume $K = 12$ hr, $x = 0.1$, and that outflow equals inflow initially. Plot the inflow and outflow hydrographs.

Time (hr)	Inflow (cfs)
6	50
12	75
18	150
24	450
30	1000
36	840
42	750
48	600
54	300
60	100
66	50

- 4.7. Using the Muskingum method, route the following inflow hydrograph assuming (a) $K = 4$ hr, $x = 0.2$, and (b) $K = 2$ hr, $x = 0.4$. Plot the inflow and outflow hydrographs for each case assuming initial outflow equals initial inflow.

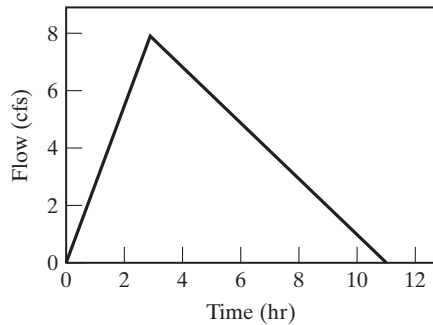
Time (hr)	Inflow (m^3/s)
0	0
2	5
4	25
6	50
8	35
10	21
12	13
14	7.5
16	2.5
18	0

- 4.8. Develop a Muskingum routing program in Excel, and repeat Problem 4.7. Compare the results.
- 4.9. Flood route the given input hydrograph through a linear reservoir ($S = KQ$), given $K = 2.5$ hr and $\Delta t = 1$ hr. Solve one step beyond the peak outflow by developing a simple relation for computing outflow Q_2 , as a function of Q_1 , and inflows I_1 and I_2 . Assume that storage and outflow are initially zero. Use the fact that $S = K(xI + (1 - x)Q)$ and $I - Q = \Delta S/\Delta t$. (*Hint:* Use the fact that the reservoir is linear to determine x).

Time (hr)	Inflow (cfs)
0	0
1	100
2	250
3	400
4	350
5	300
6	200
7	100
8	50
9	0

- 4.10.** A detention pond needs to be designed with a total capacity of 30 ac-in. of storage. The inflow hydrograph for the pond is given in Figure P4–10. Assume that that pond is initially 50% full and outflow will cease when the pond is again 50% full. Graphically determine the peak outflow from the pond assuming a linear rise and fall in the outflow hydrograph. Draw the outflow hydrograph. At what time does outflow cease? (*Hint:* The outflow hydrograph will peak at the intersection of the inflow and the outflow hydrographs. Also, make use of the fact that the available volume for storage is equal to the area between the two triangular hydrographs and must be equal to 50% of the storage.)

Figure P4–10



- 4.11.** An inflow hydrograph is given for a reservoir that has a weir-spillway outflow structure. The flow through the spillway is governed by the equation

$$Q = 3.75Ly^{3/2} \text{ (cfs)},$$

where L is the length of the weir and y is the height of the water above the spillway crest. The storage in the reservoir is governed by

$$S = 300y \text{ (ac-ft)}.$$

Using $\Delta t = 10$ hr, $L = 15$ ft, and $S_0 = Q_0 = 0$, route the inflow hydrograph through the reservoir using the storage indication method.

Time (hr)	Inflow (cfs)
12	40
24	35
36	37
48	125
60	340
72	575
84	722
96	740
108	673
120	456
132	250
144	140
156	10

- 4.12.** A reservoir has a linear S - Q relationship of

$$S = KQ,$$

where $K = 1.3$ hr. The inflow hydrograph for a storm event is given in the table.

- Develop a simple recursive relation using the continuity equation and S - Q relationship for the linear reservoir [i.e., $aQ_2 = bQ_1 + c\bar{I}$, where a , b , and c are constants and $\bar{I} = (I_1 + I_2)/2$].
- Storage route the hydrograph through the reservoir using $\Delta t = 1$ hr.
- Explain why the shape of storage-discharge relations is usually not linear for actual reservoirs.

Time (hr)	Inflow (m^3/s)
0	0
1	100
2	200
3	400
4	300
5	200
6	100
7	50
8	0

- 4.13.** Given the reservoir with a storage-discharge relationship governed by the equation

$$S = KQ^{3/2},$$

route the inflow hydrograph for Problem 4.5 using storage routing techniques and a value of $K = 1.31$ for Q in m^3/s and S in $\text{m}^3/\text{s}\cdot\text{hr}$. Discuss the differences in the outflow hydrograph for this reservoir and for the reservoir of Problem 4.12. Use $\Delta t = 1$ hr.

- 4.14.** A storm event occurred on Falls Creek Watershed that produced a rainfall pattern of 5 cm/hr for the first 10 min, 10 cm/hr in the second 10 min, and 5 cm/hr in the next 10 min. The watershed is divided into three subbasins (see Fig. P4-14), with the unit hydrographs given in the following table. Subbasins A and B had a loss rate of 2.5 cm/hr for the first 10 min and 1.0 cm/hr thereafter. Subbasin C had a loss rate of 1.0 cm/hr for the first 10 min and 0 cm/hr thereafter. Using a simple lag routing method (time shift the hydrograph by K), determine the storm hydrograph at point 2. Assume a lag time K of 40 min.

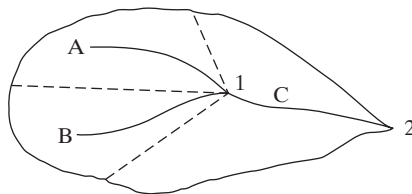


Figure P4-14

10-Minute Unit Hydrographs

Subbasin A		Subbasin B		Subbasin C	
Time (min)	Q (m ³ /s)	Time (min)	Q (m ³ /s)	Time (min)	Q (m ³ /s)
0	0	0	0	0	0
10	5	10	5	10	16.7
20	10	20	10	20	33.4
30	15	30	15	30	50.0
40	20	40	20	40	33.4
50	25	50	25	50	16.7
60	20	60	20	60	0
70	15	70	15		
80	10	80	10		
90	5	90	5		
100	0	100	0		

- 4.15.** Repeat Problem 4.14 using Muskingum routing methods instead of simple lag routing. Discuss the differences between the two storm hydrographs. Use $x = 0.2$, $K = 40$ min, and $\Delta t = 10$ min. The hydrographs in each subarea are found in Problem 4.14.
- 4.16.** Develop a spreadsheet for Muskingum routing using the given inflow hydrograph through the river reach, where $K = 3$ hr, $x = 0.15$, and $\Delta t = 2$ hr.

Time (hr)	Inflow (m ³ /s)
0	60
2	100
4	200
6	360
8	600
10	1310
12	1930
14	1460
16	930
18	650
20	440
22	300
24	180
26	120
28	80
30	60

- 4.17.** A reservoir has the storage-discharge relationship below. Route the inflow hydrograph in Problem 4.16 through the reservoir, assuming an initial storage of 52×10^6 m³ of water.

Storage (10 ⁶ m ³)	Discharge (m ³ /s)
52	20
56	120
67.5	440
88	1100
113	2000

- 4.18.** The storage equation has been given as

$$S_i = K[xI_i + (1 - x)O_i]$$

and the continuity equation as

$$\bar{I} = \bar{O} + \Delta S / \Delta t.$$

Given that I and O are the average inflow and outflow within the time period and ΔS is the change in the storage, derive the Muskingum river routing equation:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1,$$

where I_2 and O_2 refer to the inflow and outflow at the end of the time period, and I_1 and O_1 refer to those values at the beginning of the time period. Verify the equations for C_0 , C_1 , and C_2 given in Section 4.2.

- 4.19.** Determine the outflow hydrograph given the inflow hydrograph below. Use Muskingum routing, taking $K = 2$ hr, $x = 0.1$, and $\Delta t = 1$ hour.

Time (hr)	Inflow (m ³ /s)
1	100
2	140
3	200
4	320
5	440
6	560
7	750
8	700
9	600
10	500
11	400
12	300
13	200
14	100
15	100
16	100
17	100
18	100
19	100
20	100

- 4.20.** Develop a new flood routing method for rectangular cross sections based on the Muskingum and storage indication techniques. Instead of using the Muskingum storage equation $S = f(K, x, I, Q)$, use Manning's equation in the form where a and m are constants. Assume prismatic channel conditions at each time step, and derive the necessary equation for flood routing in a given rectangular channel with length L , inflows I_1 and I_2 , channel width B , and outflows Q_1 and Q_2 . Manning's equation takes the form

$$Q = ay^m.$$

- 4.21.** Equation (4-47) and Manning's equation (Eq. 4-48) can be combined to develop a second form of the kinematic wave equation [Eq. (4-56)], but in terms of Q .

- (a) Prove that the equation

$$\frac{\partial Q}{\partial x} + a\beta Q^{\beta-1} \frac{\partial Q}{\partial t} = q$$

is another form of the kinematic wave equation.

- (b) What are the values of a and β from Manning's equation?

- 4.22.** Repeat Problem 4.11 for the case that the reservoir is partially full of water with initial height above the weir of $y = 3$ ft. Assume vertical walls (i.e., constant surface area for all depths).
- 4.23.** The hydrograph at the upstream end of a river is given in the following table. Develop the hydrograph for the downstream end of this river reach according to the Muskingum–Cunge method. The reach of interest is 4 km long. The channel is trapezoidal (2:1 side slope) with bottom width of 10 m. Assume $S_0 = 0.001$, $\Delta x = 2$ km, $\Delta t = 30$ min, $c = 1.47$ m/s, and no lateral inflow.

Time (min)	Flow (m ³ /s)
0	0
30	4
60	14
90	27
120	30
150	29
180	27
210	24
240	18
270	12
300	8
330	5
360	3
390	1
420	0

- 4.24.** The inflow hydrograph to a rectangular channel is tabulated below. The channel has a width of 100 ft, a bottom slope of 0.001, and Manning n of 0.035. The length of the channel is $L = 30,000$ ft. Using the Kinematic Wave, obtain the outflow hydrograph.

Time (min)	Inflow (cfs)
0	2500
20	2500
40	3500
60	4500
80	5500
100	6500
120	5000
140	4000
160	3000
180	2000
200	2000
220	2000

- 4.25.** Using only the inflow hydrograph from Example 4.7 (i.e., no precipitation), use HEC- HMS* (<http://www.hec.usace.army.mil/software/hec-hms/download.html>) to route the hydrograph through a channel using the Muskingum–Cunge routing method. The channel has a trapezoidal cross section with a 3:1 side slope (horizontal:vertical), a Manning’s roughness (n) of 0.039, and a bottom width of 8 meters. Assume there is no loss in the channel. What is the value of peak flow and when does it occur? (*Refer to Chapter 5 for detailed information on HEC-HMS.)
- 4.26.** Repeat Problem 4.25 for reach lengths of 6, 12, and 32 km. What happens to the timing and value of the peak flow? Explain.
- 4.27.** Bull Creek Watershed has a reservoir with storage relationship

$$S = AH,$$

where A is the area (300 ac) and H is the depth or head of the reservoir in feet.

For simplicity, the area is assumed to be constant with depth. The outflow is governed by the equation

$$Q = 56.25H^{3/2},$$

where Q is in cfs. Route the storm hydrograph represented in the following table through the reservoir, using the fourth-order Runge–Kutta method.

The governing equation is

$$\frac{\Delta H}{\Delta t} = f(t, H) = \frac{Q_{\text{in}}(t) - Q_{\text{out}}(H)}{A_r(H)}.$$

The fourth-order R-K equation will be used as shown in Equations (4–20) and (4–21). The assumption is made that $H_0 = 0$ ft and $Q_{\text{out}}(H_0) = 0$ cfs. Values of Q_{in} are interpolated as necessary, and values of Q_{out} are found from the equation $Q = 56.25H^{3/2}$.

Time (hr)	Inflow (cfs)
12	40
24	35
36	37
48	125
60	340
72	575
84	722
96	740
108	673
120	456
132	250
144	140
156	10

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Chapter 5

Hydrologic Simulation Models

—Zheng Fang and George M. Doubleday



GIS & hydrologic laboratory. (Photo courtesy Donald Soward.)

5.1 INTRODUCTION TO HYDROLOGIC MODELS

Advances in computer hardware and software since the 1970s combined with larger and more extensive hydrologic data-monitoring efforts allowed for the development and application of a number of models in hydrology. These computer models can be used for a variety of purposes in simulating hydrologic response under a number of assumptions within a watershed area. Such models incorporate various equations to describe hydrologic transport processes and storages and to account for water balances in space and time. Complex rainfall patterns and heterogeneous basins can be simulated with relative ease if watershed and hydrologic data are sufficient, and various design and control schemes can be tested with hydrologic models. HEC models will be highlighted in this chapter with detailed examples, and others are presented in Chapters 6, 9, and 11.

Earlier chapters have presented a number of computational methods routinely used to convert rainfall into a storm hydrograph at the basin outlet. The rainfall–runoff process was presented in detail in Chapters 1 and 2, where we showed how input rainfall is distributed into various components of evaporation, infiltration, detention or depression storage, overland flow, and

Table 5-1 Factors Affecting the Hydrograph

- | |
|----------------------------------|
| 1. Rainfall intensity & duration |
| 2. Watershed size |
| 3. Watershed slope |
| 4. Watershed shape |
| 5. Watershed storage |
| 6. Channel type and morphology |
| 7. Channel storage |
| 8. Land use/land cover |
| 9. Soil type & infiltration |
| 10. Percent impervious |

eventually streamflow. The concept of the unit hydrograph was demonstrated converting rainfall over a watershed into associated runoff. Chapter 3 presented various statistical methods for treating return frequency events such as rainfall or streamflow hydrologic series. Chapter 4 presented various flood routing methods within channels and rivers to move hydrographs from one point to another. Hydrologic models allow for hydrologic prediction in space and time through the use of well-known numerical methods.

The actual shape and timing of the response hydrograph for a given watershed have been shown to be a function of many physiographic, land use, and meteorological variables, which are listed in Table 5-1. Rainfall intensity and duration are the major driving forces of the rainfall-runoff process, followed by watershed characteristics that translate the rainfall input into an output hydrograph at the outlet of the basin. Size, slope, shape, soils, and storage capacity are all important parameters in watershed geomorphology. Land use and land cover parameters can significantly alter the natural hydrologic response through increases in impervious cover, altered slopes, and improved drainage channel networks.

Major research programs by Harvard University, Stanford University, and the U.S. Army Corps of Engineers during the 1960s were pioneering efforts to use early versions of digital computers to simulate watershed behavior on a continuous basis. The Stanford Watershed Model (Crawford and Linsley, 1966), which later evolved into HSPF (Hydrologic Simulation Program—FORTRAN) (Johanson et al., 1980; US EPA, 2007), was the first available major watershed model. Continuous models such as HSPF are based on long-term water-balance equations and thus account directly for effects of antecedent conditions.

From these early efforts, a range of modeling approaches including event models was developed and applied during the 1970s for urban stormwater, floodplain hydrology, agricultural drainage, reservoir design, and river basin management. Much of the model development was driven by the interest in stream and lake water quality in urban runoff that surged during the 1970s.

Single event models are designed to simulate rainfall–runoff from single storm events. Models such as the original HEC-1 Flood Hydrograph Package (HEC, 1981), the revised HEC-HMS (HEC, 1998, 2006, 2010) are widely used for floodplain analysis. The more sophisticated model was the Storm Water Management Model (SWMM), developed in 1971 for EPA to address in detail the quantity and quality variations in urban runoff (Metcalf et al., 1971). The SWMM model can be used for storm event or continuous simulation and has been through a number of updates and improvements over the years (Huber and Dickinson, 1988; SWMM5, 2005; Rossman, 2010). This model is considered one of the most comprehensive for simulating urban runoff dynamics, drainage design, hydraulic analysis, and nonpoint source water quality. With more demands from end users, the SWMM model was enhanced into a commercial modeling package—XP-SWMM (eXPert Stormwater and Wastewater Management Model) by XP Software (2010). Besides all the capabilities of SWMM, the XP-SWMM model has a better graphical user interface and the ability of linking the 1-D flow simulation to a 2-D overland flow model to generate floodplain maps.

As an important addition to the water resources modeling field, the Danish Hydraulic Institute (DHI) Water and Environment, Inc., provides a whole suite of hydrologic and hydraulic modeling tools (MIKE URBAN, MIKE FLOOD, etc.) to address engineering problems of rainfall–runoff, pipe flows, overland flows, water quality, and sediment transport. For instance, the MIKE URBAN model enables users to simulate water, wastewater, and storm water in one integrated GIS-based framework that is powered by ESRI's ArcObjects components (DHI, 2011). The basic module of MIKE URBAN - Model Manager incorporates the U.S. EPA's fully dynamic storm water and waste water network modeling package (SWMM 5) and the standard distribution network model (EPANET) to simulate hydraulics and water quality. In addition, the 2-D flow simulation module provides an easy and flexible tool for computing and visualizing the flows above and below ground in one fully integrated dynamic simulation.

Hydrologic simulation models can be classified according to a wide range of characteristics, as shown in Table 5–2. For watershed analysis, the major categories of interest include lumped parameter vs. distributed parameter, event vs. continuous, and stochastic vs. deterministic. Lumped

Table 5–2 Hydrologic Models

Model Type	Example of Mode
Lumped parameter	HEC-HMS, Snyder or Clark UH
Distributed	Vflo, MIKE URBAN, MIKE FLOOD
Event	HEC-HMS, SWMM, SCS TR-20
Continuous	SWMM, HSPF, Vflo
Physically based	HEC-HMS, SWMM, HSPF, Vflo
Stochastic	Synthetic streamflows

parameter models transform actual rainfall input into runoff output by conceptualizing that the subwatershed processes occur at one spatial subarea (as in a “black box”). Model parameters may or may not have a direct physical definition in the system. Synthetic unit hydrographs (UHs) presented in Chapter 2 are a widely used method to convert rainfall into runoff within a watershed or basin.

Distributed parameter models attempt to describe physical processes and mechanisms in space, as evidenced by certain classes of hydrologic simulation models. Kinematic wave methods have become very popular to compute both overland flow and channel routing. This is due to heightened interest in distributed hydrologic modeling with the advent of GIS and digital elevation data for watersheds that has become available since the mid-1990s. These issues are described in more detail in Chapters 10 and 11 (i.e., Vflo) along with radar rainfall input and digital elevation methods for setting up grids to compute overland flow and channel flow.

Both HEC and SWMM rely on the kinematic wave methods described in Chapter 4. Major new graphical user interfaces (GUIs) have been developed for these two models and have revolutionized the way in which we deal with hydrologic data and parameter inputs. The HEC-HMS and its 2010 version 3.5 are presented in this chapter, the HEC-RAS model is presented in detail in Chapter 7, and the SWMM model is described with a detailed example (SWMM5) in Chapter 6. A useful guide to the EPA SWMM5 model has been compiled and updated by Rossman (2010).

Some models include a random or stochastic component to represent input rainfall, which is then used to generate time series of streamflow. The time series can then be statistically evaluated using flood frequency analysis. Hydrologic synthesis allows hydrologists to extend short historical records, such as streamflow, to longer sequences based on statistical methods. Synthetic sequences either preserve the statistical character of the historical record or follow a prescribed probability distribution, such as a lognormal or log Pearson type 3 (Section 3.5). Mass curve analysis assumes that historical records will repeat exactly. Random generations assume that successive flows are independent and distributed according to a known probability distribution. Details on stochastic models can be found in Bras and Rodriguez-Iturbe (1984), Stedinger et al. (1993), and Hirsch et al. (1993).

A major advantage of simulation models is the insight gained by gathering and organizing data required as input to the mathematical algorithms that comprise the overall model system. This exercise can often guide the collection of additional data or direct the improvement of mathematical formulations to better represent watershed behavior. Another advantage is that many alternative schemes for water supply systems, for urban development, or flood control options can be quickly tested and compared with simulation models.

The major limitation of simulation models is the inability to properly calibrate and verify applications in which input data are lacking. Current practice assumes that the simplest model that will satisfactorily describe the

system for the given input data should be used. Model accuracy is largely determined by available input data and observed input and output time series at various locations in a watershed. Modern radar rainfall, hydrologic, and topographic datasets are now available for many areas, and model accuracy has increased accordingly.

Despite their limitations, simulation models still provide the most logical and scientifically advanced approach to understanding the hydrologic behavior of complex watershed and water resources systems. The availability of digital elevation datasets and radar rainfall data has opened a new era in the science of hydrology and has led to many new design and operating policies never realized or tested before (see Chapters 6, 9, 10, and 11). Several excellent reviews of models in hydrology have been published, including Maidment (1993), DeVries and Hromadka (1993), Hoggan (1997), James and James (1998a), and McCuen (2005). The reader is referred to these references for details beyond the scope of this chapter.

5.2
STEPS IN
WATERSHED
MODELING

With so many hydrologic models available to the hydrologist or civil engineer, very little new model development is currently being supported. Rather, one must select one of the available simulation models based on characteristics of the system to be studied, the objectives to be met, and the available budget for data collection and analysis. Once the model is selected, the steps involved in watershed simulation analysis generally follow the sequence of Table 5–3.

Table 5–3 Steps in Watershed Modeling

1. Select model based on study objectives and watershed characteristics, availability of data, and project budget. 2. Obtain all necessary input data—rainfall data, digital topography, land use and soils, infiltration, channel characteristics, streamflow data, design floods, and reservoir data. 3. Evaluate and refine study objectives in terms of simulations to be performed under various watershed conditions. 4. Choose methods for determining subbasin hydrographs and channel routing. 5. Calibrate model using historical rainfall, streamflow, and existing watershed conditions. Verify model using other events under different conditions while maintaining same calibration parameters. 6. Perform model simulations using historical or design rainfall, various conditions of land use, and various control schemes for reservoirs, channels, or diversions, to the extent possible. 7. Perform sensitivity analysis on input rainfall, routing parameters, and hydrograph parameters as necessary. 8. Evaluate usefulness of the model and comment on needed changes or modifications.

The selection of a model is a very difficult and important decision, since the success of the analysis hinges on accuracy of the results. Table 5–2 should be considered along with characteristics of the watershed, study objectives, and so on to develop a modeling plan. In general, unless digital watershed data are extensive in space and time, the usual approach to watershed analysis is to use a deterministic event model with lumped parameter concepts for developing hydrographs and flood routing. The HEC-HMS (2006, 2010) is one of the most widely used models for typical watershed analysis to convert rainfall to runoff and streamflow. An Applications Guide was written in 2008 by Ford et al. to illustrate applications of HEC-HMS to studies on urban floods, flood frequency, flood loss reduction, flood warning, reservoir design, and environmental issues.

If a watershed has extensive data available on rainfall, infiltration, baseflow, streamflow, and soils and land use, then HSPF from U.S. EPA BASINS (2007) can be applied for calculating continuous long-term water balances and outflow hydrographs. For the case of a well-defined urban drainage network, a distributed event model such as SWMM (Huber and Dickinson, 1988; James and 1998b; SWMM5, 2005; Rossman, 2010) can be applied to define hydrologic response for components throughout the system. Urban stormwater models are discussed in more detail in Section 5.3 and Chapter 6. Another GIS-based distributed model, Vflo, is presented in Chapter 11.

Step 5 in Table 5–3, model calibration and verification, is important in fitting the model parameters and producing accurate and reliable results in steps 6 and 7. Model calibration involves selecting a measured set of input data (rainfall, channel routing, land use, and so on) and measured output hydrographs for model application. The controlling parameters in the model are adjusted until a “best fit” is obtained for this set of data. The model should then be “verified” by simulating a second or third event (i.e., different rainfall) and keeping all other parameters unchanged to produce a comparison of predicted and measured hydrographs. A detailed example of the HEC-HMS model is described in Section 5.6, where an actual case study is highlighted to indicate difficulties and complexities that are often encountered in watershed analyses.

A selected number of the most popular event, continuous, and urban runoff models for hydrologic simulation are listed in Table 5–4. Universities or federal agencies supported the development of most of these models. Some of the models are very well documented and, as a result, have seen wide application to watersheds, especially in the United States. Others have been applied only to specific areas in the country. Extreme care and judgment must be exercised in applying any one of these models to areas where data exist to define UH and routing parameters.

5.3 DESCRIPTION OF MAJOR HYDROLOGIC MODELS

Several of the surface water models in Table 5–4 will be briefly reviewed below.

The **HSPF** was the commercial successor to SWM-IV and was modified to include water quality considerations, kinematic wave routing, and variable time steps. The current version is HSPF, which is a redesigned modular version that performs simulations of a variety of hydrologic and water quality processes on or under the land surface, in channels, and in reservoirs (Johanson et al., 1980). The HSPF model simulates both watershed hydrology and water quality. It allows an integrated simulation of land and soil contaminant runoff processes with in-stream hydraulic and sediment-chemical interactions. The program provides a time history of runoff rate, sediment load, and nutrient and pesticide concentration, along with a time history of water quality and quantity at specific points in a watershed. HSPF computes a continuous hydrograph of streamflow at the basin outlet. Input is a continuous record of rainfall and evaporation data. Infiltration is divided into surface runoff and interflow, and deeper percolation to the ground water. Three soil moisture zones are included in the model. Total streamflow is a combination of overland flow, interflow, and ground water flow.

One of the most useful modules in HSPF is the nonpoint-source model (NPS), which provides for continuous simulation of nonpoint pollutants from urban and undeveloped land surfaces. It is available as a separate model from EPA and has been extensively tested with HSP by the Northern Virginia Planning Commission. HSPF is the model of choice for evaluation of continuous runoff and nonpoint-source loads (EPA-BASINS, 2007). Originally introduced in 1996 with subsequent releases in 2001 and 2007, BASINS is a multipurpose environmental analysis system designed for use by regional, state, and local agencies in performing watershed and water-quality-based studies. This system makes it possible to quickly assess large amounts of point-source and nonpoint-source data in

Table 5–4 Selected Simulation Models in Hydrology

Model	Author	Date	Description
HEC-HMS	HEC	2006, 2010	Hydrologic modeling system (replacing HEC-1)
HEC-RAS	HEC	2006, 2010	River analysis system (replacing HEC-2)
SCS-TR20	USDA SCS	1984	Hydrologic simulation model
HSPF	EPA Basins Version 4.0	2007	Hydrological simulation program—Fortran
SWMM	Huber and Dickinson	1971, 1988, 2005	Storm water management model
MIKE URBAN	DHI	2011	Modeling and GIS for wastewater and storm water collection systems
MIKE FLOOD	DHI	2011	Combined river, sewer, and floodplain modeling
DWOPER	NWS, Fread	1978	NWS operational dynamic wave model
UNET	Barkau	1992	One-dimensional dynamic wave

a format that is easy to use and understand. Installed on a personal computer, BASINS allows the user to assess water quality at selected stream sites or throughout an entire watershed. This invaluable tool integrates environmental data, analytical tools, and modeling programs to support development of cost-effective approaches to watershed management and environmental protection.

The **EPA SWMM** development dates back in the 1970s. SWMM is the most comprehensive urban runoff model and provides for continuous and/or event simulation for a variety of catchments, conveyance, storage, treatment, and receiving streams. Both water quantity and quality can be simulated for a stormwater network, and flow routing can be performed by nonlinear reservoir methods, kinematic wave methods, or with the full Saint Venant equations in the SWMM EXTRAN Block. The SWMM hydraulic engine is able to handle gradually varied, one-dimensional unsteady flow throughout the drainage network taking account of backwater effects, reversed flow, surcharged flow, pressurized flow, and interconnected ponds. The model has gone through a number of revisions; version 5 (Huber and Dickinson, 1988; James and James, 1998b; SWMM5, 2005; Rossman, 2010) is currently available.

The new **XP-SWMM** was released in 1997 and contains a 2-D model for simulating flows on the surface combined with the full pipe network analysis from the original model. Scientists and engineers often use XP-SWMM to develop link-node and spatially distributed models to assess, design, and simulate flows in storm water and wastewater systems and in natural systems such as river, lakes, and floodplains with groundwater interaction. Hydraulically, flows are simulated in 1-D channels and pipes, and can be coupled to a powerful 2-D engine for comprehensive flood modeling and mapping. Like other hydrologic models, XP-SWMM simulates the complete hydrologic cycle for rural and urban watersheds starting from rainfall events through collection, conveyance, and treatment systems to the final outfalls. The model takes into account snowmelt, evaporation, infiltration, surface ponding, and ground-surface water exchanges in the process of hydrologic/hydraulic simulations. With the water quality module, XP-SWMM is able to route pollutants and treatment through the urban and natural systems. Currently, XP-SWMM has been approved by the U.S. Federal Emergency Management Agency (FEMA) as a computer model in floodplain mapping for the National Flood Insurance Program (NFIP) and has been popularly used in the fields of storm water management, sanitary sewers, floodplain management, hydrology, hydraulics, and water quality. SWMM is described in full detail with examples in Chapter 6.

As one of the many hydrologic models that provide distributed simulation and hydrologic prediction, Vflo, a physics-based model developed by Vieux & Associates, Inc., allows for spatial variability to be considered within a watershed when calculating runoff. It uses input from the GIS that provides gridded watershed data for a variety of parameters, such as elevation, land

use, and soils. Within Vflo, each grid cell can be assigned different properties that allow a watershed with highly diverse characteristics to be modeled precisely (Vieux, 2004). The kinematic wave analogy is used to route runoff from overland flow to channel flow, according to a flow direction grid derived from a digital elevation model (DEM). Vflo has been proven performance based on extensive applications in low-slope urbanized watersheds (Fang et al., 2010). The model takes into account overland flow, infiltration, and channel flow in a connected grid system built within a governing GIS environment. Vflo shows many advantages, including its flexibility for data setup, its accuracy in hydrograph comparison, its ability to handle NEXRAD rainfall directly, and the fact that one can observe hydrographs at any location within the watershed. Readers can find more information and examples of Vflo in Chapters 10 and 11.

The demands for better tools for hydrologic and hydraulic modeling have inherently increased, and there is an irreversible trend toward a combined 1-D and 2-D approach in floodplain and urban flooding studies. DHI Water & Environment, a research and consulting organization, developed MIKE series of numerical modeling software and applied advanced technologies in hydraulic and water resources engineering since 1964. For better understanding of flow dynamics in urban settings, people often use MIKE URBAN and MIKE FLOOD. **MIKE URBAN** is a GIS-based modeling tool for water distribution and wastewater collection systems based on three major modules: MIKE URBAN Model Manager, Collection System (CS), and Water Distribution (WD). Because the MIKE URBAN Model Manager provides a complete modeling environment with the latest versions of public domain modeling tools—SWMM 5 and EPANET to simulate dynamics of flows in storm water and wastewater systems, it can handle simulations of flows in sewers, storm water drainage systems interacting with 2-D overland flows, and flows in water distribution systems as well.

MIKE FLOOD is another computer program developed by DHI that simulates river inundation, floodplains, and urban drainage systems. It dynamically couples 1-D (MIKE 11 and MOUSE) and 2-D (MIKE 21) features into one integrated numerical modeling tool, which enables people to simulate different flood problems related to rivers, floodplains, floods in streets, drainage networks, coastal areas, dam operations, and levee breaches. Thus, MIKE FLOOD is suitable to handle flood assessments, flood hazard mapping, flood risk analysis, and integrated urban drainage design. For more detailed information, readers should consult with the DHI Water & Environment (DHI, 2011).

The **SCS** or Natural Resources Conservation Service (**NRCS**) has developed a number of methods and models that are based on its extensive small-watershed database. Graphical, tabular, and chart methods are available for the SCS TR-55 (SCS, 1986), entitled “Urban Hydrology for Small Watersheds.” These methods are based primarily on the runoff-curve-number method described in Chapter 2. The SCS TR-20 (SCS, 1984) is a computerized

method for solving hydrologic problems using SCS procedures. The program develops runoff hydrographs, flood routes these through channel reaches and reservoirs, and combines or separates hydrographs at confluences. For each subbasin, the area, runoff curve number, and time of concentration are required. Routing procedures are somewhat simpler than those used in HEC-HMS and include the convex routing method or a simple routing coefficient. NRCS methods are often used for agricultural and undeveloped areas or forested sites. The interested reader should consult McCuen (2005) for discussion and application of NRCS methods.

The preceding discussion has provided a review of many of the popular hydrologic simulation models used for watershed analysis. The reader should consult user documentation directly from authors before attempting to apply the models to an actual watershed. Many new models have been released in response to availability of radar rainfall, digital elevation data, and faster computer technology.

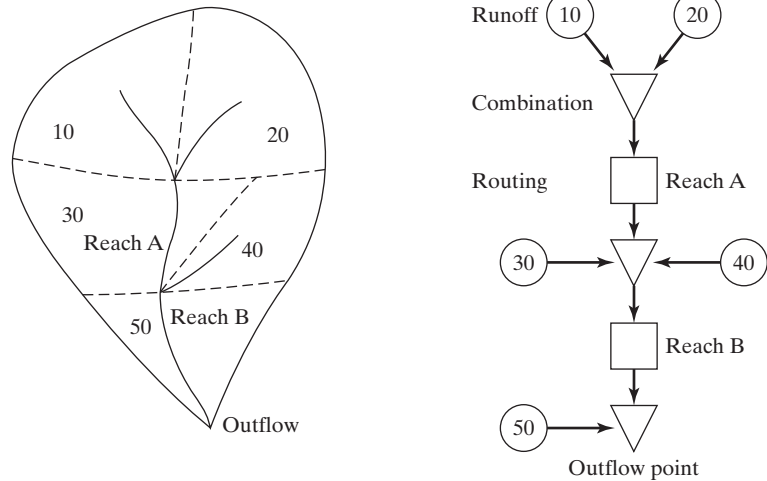
The next two sections are devoted to introduction to HEC-HMS theory (Section 5.4) and application (Section 5.5), along with examples. It is considered by many to be the most versatile model and is the most often used of the available computer models described. The current HEC-HMS was released in 2010 (HEC, Version 3.5) and is presented along with detailed case studies in Sections 5.5 and 5.6. HEC-HMS offers significant advantages to the first-time user over other models described above, since the GUI has been very well designed. The model is a very useful educational tool for any first-time student to the field of hydrologic models, and continues to be improved and updated. New students should spend time learning about HEC-HMS, since it is available free over the Internet, and details on its availability and related hydrologic training courses are provided in Appendix E.

HEC-1 was originally designed in 1967, with the official release in 1973, to simulate surface runoff processes from precipitation over a watershed. The flood hydrograph package was developed over a number of years by the Hydrologic Engineering Center. The student should consult the latest HEC-HMS user's manual for more details (HEC-HMS, version 3.5, 2010).

The process of converting precipitation to direct runoff can be simulated for either small subbasins or large complex watersheds. An original HEC model for any river basin has basic components for subbasin runoff, channel and reservoir routing, and hydrograph combining (see Fig. 5–1). Subarea boundaries are delineated so that lumped precipitation loss and watershed parameters can be used. The computations proceed from upstream to downstream in the watershed, and hydrograph data or plots can be provided at any convenient point. Historical rainfall events or design rainfall is transformed to runoff via UH methods covered in Chapter 2. A discharge hydrograph is computed at the outlet of each subarea

5.4 HEC-HMS FLOOD HYDROGRAPH THEORY

Figure 5-1
HEC model
configuration.



(see Fig. 5-1). For example, a storm hydrograph for a given rainfall would be computed for subareas 10, 20, 30, 40, and 50 in Figure 5-1.

The routing component in HEC-HMS requires input parameters to define the specific routing characteristics of a river reach or reservoir. Output consists of an outflow hydrograph at the downstream station. Hydrograph combining at key locations is essential for the overall system logic in HEC and allows for an optimal use of computer storage in the model. Combine steps occur at the triangle indicators in Figure 5-1, where two or more hydrographs are joined. Finally, simple river routing as presented in Chapter 4 is accomplished for both reach A and reach B, as shown by the square indicators. The final output hydrograph is computed at the outlet or outflow point.

HEC-HMS is a general flood hydrograph package with the following capabilities:

1. Simulation of watershed runoff and streamflow from historical or design rainfall
2. Determination of flood flows at various locations for use in flood-plain studies
3. Evaluation of land use or topographic changes in a watershed area
4. Simulation of flood control measures, such as reservoir storage and channel options
5. Computation of damage frequency curves and expected annual damages for various locations and multiple flood control plans

Watershed Delineation and Organization

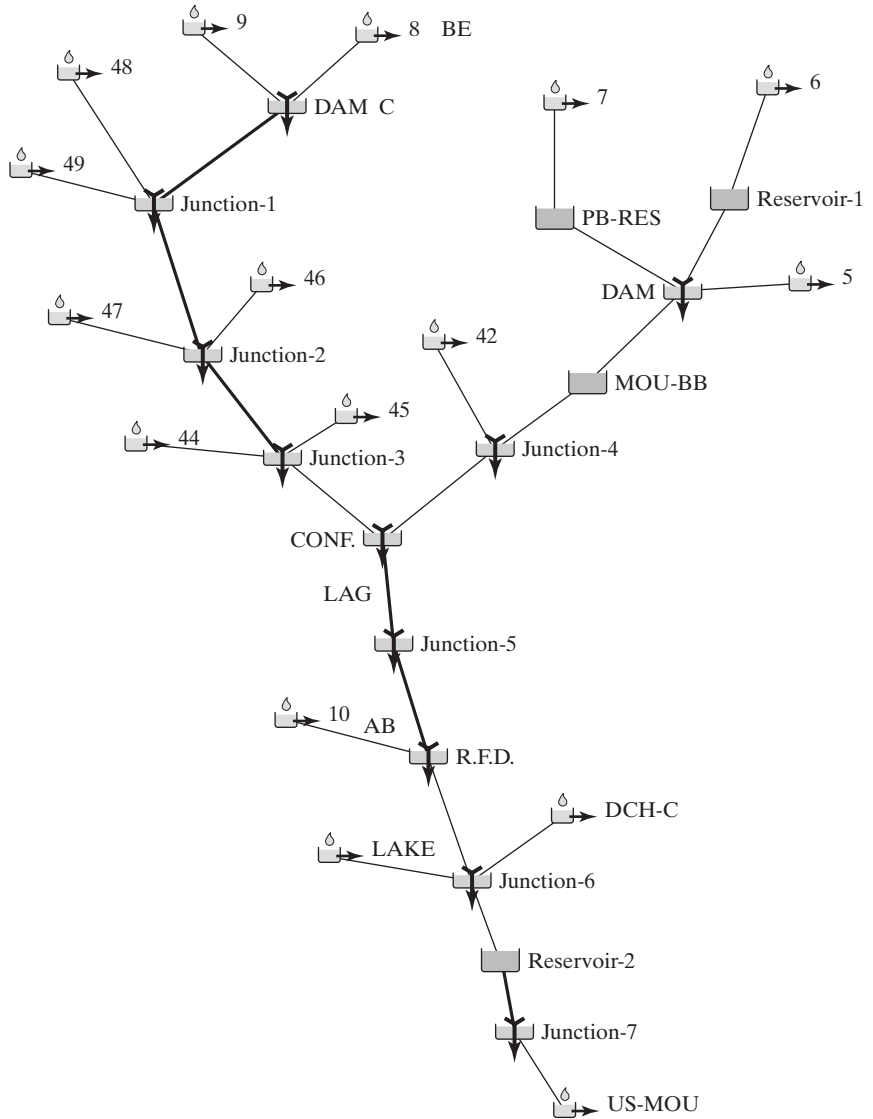
HEC-HMS uses parameters averaged in space and time to simulate the runoff process. The size of subbasins, routing reaches, or computation interval is selected based on the basin physiography, available rainfall data, available streamflow data, and required accuracy. A watershed is subdivided into small and relatively homogeneous subbasins according to drainage divides based on mapped or digital topography, as shown in Figure 5–1. The size of a subbasin should generally be in the range of 1–10 mi² because of limitations of UH theory, especially in urban areas. Routing reaches are identified, and the overall order of the runoff computation is defined (from upstream to downstream) for input to HEC. Routing reaches should be long enough so that a flood wave will not travel faster than the computation time step. Otherwise, numerical errors in flood calculations will occur. Simple routing methods described in Sections 4.1 to 4.3 are contained in HEC-HMS.

Precipitation

Precipitation P is computed for each subbasin in Figure 5–1 using historical data or synthetic design storms. The model allows for (1) incremental P (depth of P for each time interval) for each subbasin, (2) total cumulative P and a time distribution, or (3) historical gage data together with areal weighting (Thiessen) coefficients for each subbasin. Standard design storms can be used in the form of (1) depth vs. duration data, (2) probable maximum precipitation, or (3) standard project precipitation. The precipitation data must be input at a constant time interval, but this interval may be different from the computational time interval in the model. Precipitation inputs are described for HEC-HMS in Section 5.5, and a detailed example is contained in Section 5.6 for a complex watershed. Figure 5–2 shows a typical basin model for the HEC-HMS model indicating various subareas and river reaches that connect the subareas together.

HEC-HMS is capable of simulating snowfall and snowmelt. Up to 10 elevation zones of equal increments may be specified in each subbasin. Usually, an elevation increment of 1000 ft is used, but any increment may be specified as long as the air-temperature lapse rate corresponds to the change in elevation within the zone. Snowmelt occurs when the temperature is equal to or greater than the melt temperature and is calculated by either the degree-day method or an energy budget method (see Section 2.9). For more detail on snowfall and snowmelt calculations, see the HEC user's manual (HEC, 2010) and *Runoff from Snowmelt* (U.S. ACOE, 1960). Dingman (2002) covers snowmelt in more detail with examples.

Figure 5-2
Example of a
watershed in
HEC-HMS.



Loss Rate Analysis

HEC contains several methods for computing the loss of precipitation to interception and infiltration, as shown in Table 5-5. The simplest is the initial and constant loss rate function, in which the initial loss volume is satisfied before the constant loss rate begins. The remaining constant loss is identical to the ϕ index method for infiltration (see Section 2.7) and is often used in design storm analysis or where data are insufficient to allow calculation with more sophisticated methods.

Table 5–5 HEC-HMS Loss Methods

Method	Description
Initial and constant	Initial loss volume is satisfied and then constant loss rate begins.
HEC exponential	Loss function is related to antecedent soil moisture condition and is a continuous function of soil wetness.
SCS curve number	Initial loss is satisfied before calculating cumulative runoff as a function of cumulative rainfall using SCS methods (Section 2.3).
Holtan method	Infiltration rate is computed as an exponential function of available soil moisture storage from Holtan's equation.
Green and Ampt	Infiltration rate is computed from the Green and Ampt equation as a function of soil moisture and hydraulic conductivity (Section 2.8).
Deficit/Constant	First an initial deficit storage is filled, and then infiltration rates can be specified on a monthly basis.
SMA	Soil moisture accounting assigns a value of initial storage to all layers of the ground using a gridded method.

The SCS method employs the SCS curve number, CN, which is directly related to land use and hydrologic soil properties of a watershed. The SCS approach relates accumulated rainfall excess or runoff to accumulated rainfall with CN (see Section 2.5). It is popular because of its application to ungaged areas and its large empirical database. The method requires input of CN, initial abstraction, and percent imperviousness (McCuen, 2005).

The Green and Ampt method was added to HEC in 1990 and probably is one of the best estimates for infiltration, since the parameters are all measurable and directly related to soil types in a watershed. Version 3.5 of HMS uses four different parameters to apply the method; it is necessary to measure or estimate hydraulic conductivity, capillary suction at the wetting front, initial moisture content, and saturated moisture content (see Section 2.8 for Green and Ampt and Table 2–4 for data). These parameters can be estimated effectively using physical soil characteristics, such as soil texture class and major soil horizon classification. Calculation of these parameters assumes that the soil is a homogeneous medium with uniform initial moisture content. The Green and Ampt method has received significant attention in recent years with the release of HEC-HMS. For example, the TSARP program instituted in Houston, TX, after T.S. Allison flooding utilized the Green and Ampt methods in modeling 22 different watersheds (see Appendix E).

Subbasin Runoff Calculation

Several methods for surface runoff or transform computations in HEC are available in the model and can be selected by the user. These are presented in Table 5–6 and include the UH methods of Clark *TC & R* (1945), Snyder (1938), and the Soil Conservation Service (SCS, 1984, 1986). Known UHs

Table 5-6 Surface Runoff Methods in HEC-HMS

Unit hydrograph input directly
Clark hydrograph method (<i>TC</i> & <i>R</i> method) (Section 2.3 and Table 5-10)
Snyder unit hydrograph method (see Section 2.3)
SCS method (CN method + SCS UH, Section 2.3)
Kinematic wave for overland hydrograph (see Section 4.6)
ModClark
User-specified S-graph

can also be directly input. The kinematic wave overland flow computation was added to HEC in 1981 and allows more accurate representation of urbanized areas for UH calculations (HEC, 1981). Kinematic wave methods are described in more detail in Section 4.6. A similar setup for conversion of rainfall to runoff exists for HEC-HMS (Section 5.5).

Clark's method is based on the time-area curve method described in Chapter 2. The time-area histogram, determined from isochrones of the watershed, or contours from points at which water arrives at the same time, is convoluted with a unit design storm hyetograph to yield the hydrograph, which is then routed through linear reservoir storage to allow for peak attenuation. The Clark-method parameters are the time of concentration and the storage coefficient *R*, defined in Section 2.5. Hoggan (1997) provides more details on the Clark method, and Table 5-10 presents empirical *TC* and *R* equations derived for Houston, TX (within Harris County).

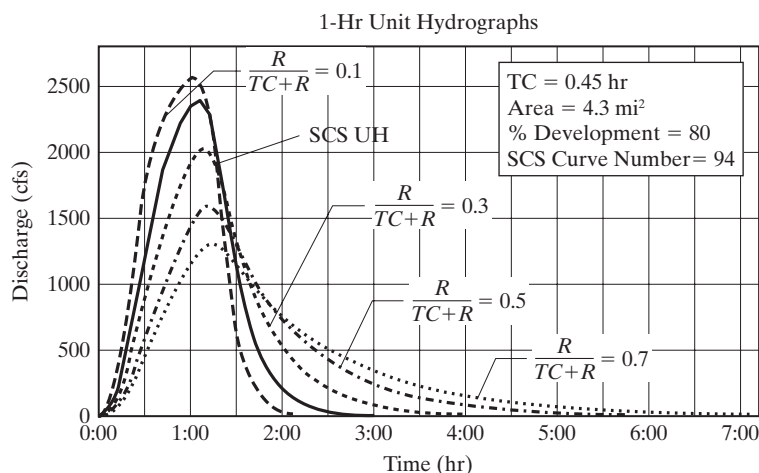
The HEC model uses a synthetic time-area curve derived from a generalized basin shape, and the equations are applicable to most basins:

$$AI = 1.414T^{1.5}, \quad 0 \leq T < 0.5, \quad (5-1)$$

$$1 - AI = 1.414(1 - T)^{1.5}, \quad 0.5 < T < 1. \quad (5-2)$$

where *AI* is the cumulative area as a fraction of subbasin area and *T* is fraction of time of concentration, *T_c*. Specific time-area curves can also be input to HEC model if they can be computed from some other GIS-type method.

The computed hydrograph based on a time-area method is routed through a linear reservoir (Section 4.3) to produce the final UH. A family of 2-hr UHs generated by HEC is shown in Figure 5-3 for a 4.3-mi² basin with *TC* = 0.45 hr. Note the wide range of hydrograph shapes that can be produced by the two-parameter UH. An SCS UH is shown for comparison. The larger the value of *R*/(*TC* + *R*), the flatter the UH response, more typical of natural watersheds with flat slopes. Thus, the smaller ratios correspond to more urbanized watersheds. The steepest hydrograph could also represent a basin with steep slopes and a small amount of storage (Hoggan, 1997).

**Figure 5-3**

Family of 1-hr UHs for a 4.3-square-mile basin with 80% development and an SCS Curve Number of 94.

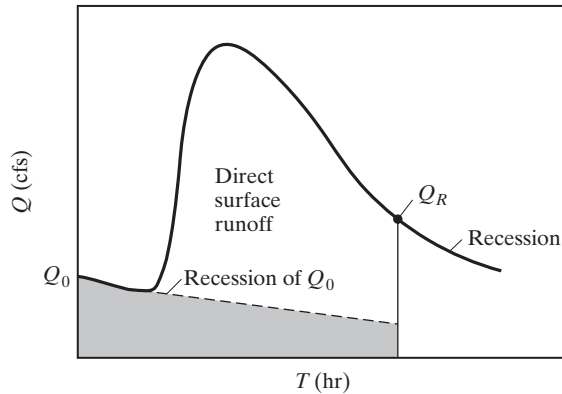
Snyder's method (Section 2.3) provides time to peak t_p and peaking coefficient C_p , which are insufficient to produce the curved shape of the hydrograph. Clark's method is therefore used to help smooth and define the shape of the hydrograph corresponding to Snyder's coefficients within the HEC model. The direct runoff ordinates are then calculated by convolution of the UH with the net precipitation for the subbasin.

The kinematic wave process, described in more detail in Section 4.6, represents a nonlinear runoff response compared with linear UH methods. The kinematic wave method relies on parameters that are generally measurable from a basin, such as slope, land use, lengths, channel shape, roughness, and area. Overland flow, collector, and main channel elements are used to represent the characteristics of a drainage basin (Fig. 4-9). In the method, the main equations are Manning's equation of flow and continuity [Eqs. (4-47) and (4-48)]. These are solved numerically to produce overland flow as a function of time and space. The same equations are solved for channel elements (Fig. 4-9). The kinematic wave (KW) method is more applicable to the analysis of urban basins, since KW theory does not provide for the attenuation of flood waves. The area of the subbasin is required along with L , S , n , shape, and size, and any upstream hydrograph to be routed in the reach. KW routing is also available in the main channel. Chapter 11 presents a section on distributed modeling with VFLO using kinematic wave methods (Vieux, 2004).

Baseflow Calculation

HEC-HMS simulates baseflow of a flood hydrograph with three parameters: (1) Q_0 , flow in channel prior to start of rising limb; (2) ratio of recession flows at $t=1$ (RTIOR), exponential decay rate; and (3) Q_R , point on falling

Figure 5-4
Components of
streamflow
hydrograph.



limb called the recession threshold (Fig. 5-4). The above parameters can be obtained through a semilog plot of observed Q vs. time. The decay rate RTIOR is equal to the slope of the line, and Q_R/Q_{peak} is usually in the range of 0.05 to 0.15.

Equation (5-3) defines the parameters that are used in the model, in which the recession flow threshold Q_R and the decay constant RTIOR must be specified by the user. Figure 5-4 defines the relation between the streamflow hydrograph and user-defined parameters. Equation (5-3) computes recession flow Q as

$$Q = Q_0(\text{RTIOR})^{-n\Delta t}, \quad (5-3)$$

where

Q_0 = starting baseflow prior to the rising limb,

Q = baseflow rate at end of $n\Delta t$,

RTIOR = ratio of recession flows at $t = 1$ hr increment apart.

Many applications of HEC are for major urban flood events, where baseflow is a relatively small percentage of the total hydrograph flow. Of course, baseflow can be entered as a constant value or as zero. Several possibilities exist for computing baseflow in HEC-HMS (Section 5.5).

Flood Routing

Flood routing involves analyzing the movement of a flood wave as it travels from reach to reach through a stream or river. Table 5-7 lists the major methods of flood routing that are available in HEC-HMS (see Chapter 4), where both storage-indication reservoir and river routing methods were covered. The Muskingum method requires a K and x , where K can be obtained from analyses shown in Section 4.2 or using the reach length divided by the wave velocity determined from Equation (4-38). dQ/dy can be obtained

Table 5-7 Flood Routing in HEC-HMS

Method	Description
Muskingum	Storage coefficient (x) plus travel time (K) through each reach
Modified Puls	Table of storage vs. outflow for each reach based on HEC-RAS water surface profile information—various storages are plotted vs. peak flows in the reach (Chapter 4)
Kinematic wave	Channel shape, length, slope, and n ; outflow from each reach based on depth of flow in continuity equation and Manning's equation (see Chapter 4)
Muskingum–Cunge 8-point	Same equations and technique as the Muskingum–Cunge but channel is described with 8 station-elevation values
Lag	Direct lag of flow through channel with translation accounted for but not attenuation

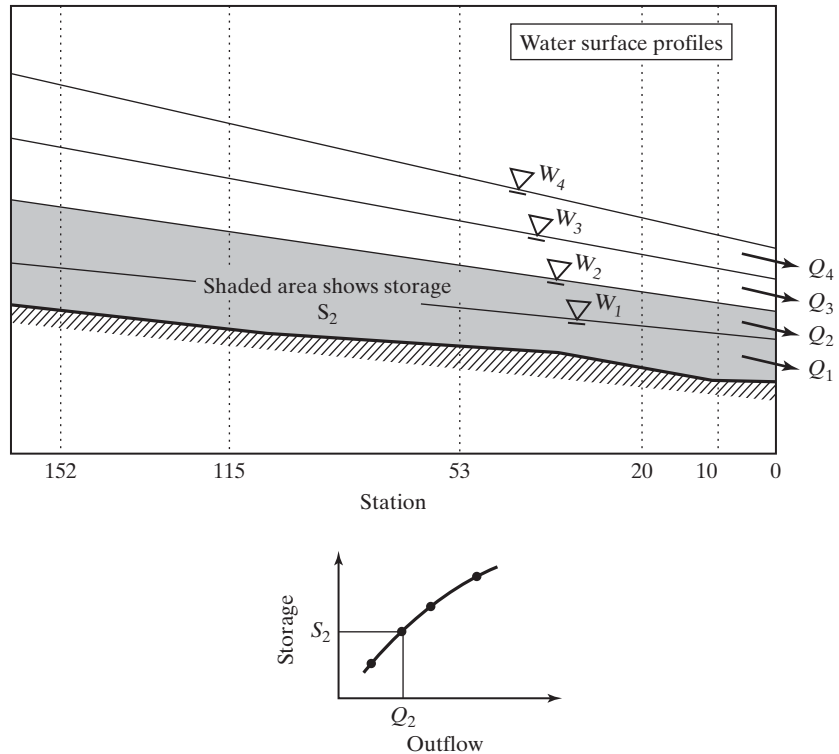
from the slope of the rating curve at a typical discharge value to be routed. To avoid numerical instability, note that for the Muskingum method, $2Kx < \Delta t \leq K$ and number of subreaches $= K/\Delta t$, where K = travel time and Δt = time interval.

The Modified Puls method was briefly presented in Section 4.3 for a reservoir but can also be applied for rivers by developing a storage–discharge S - Q relation. One of several methods can be used: (1) steady-flow profile computation, (2) observed profiles, (3) normal depth profiles, (4) storage from inflow and outflow hydrographs, and (5) optimization techniques. These are described in more detail in Hoggan (1997) and will not be repeated here. For method 1, several water surface profiles are generated: WS_1 to WS_5 , corresponding to discharges, and Q_1 to Q_5 , using the HEC-RAS program (Chapter 7). The storage volumes are computed from cross-sectional areas of the channel and their lengths for each WS_i . An S - Q curve (rating curve) is determined (Fig. 5-5). The number of routing steps to be used for a river reach is NSTEPs, equal to $K/\Delta t$. This is a numerical parameter related to flood wave speed vs. the computational time step so that numerical stability is maintained. The greater the storage in a reach per unit of flow, the greater the attenuation of the flood wave as it is routed through (see Section 5.6 for detailed example).

The kinematic wave channel routing method does not allow flood peak attenuation, which is a problem where the floodplain is extensive. Thus, KW can overestimate flows compared to other methods. But it has the advantage of allowing for laterally distributed inflow along the main channel (HEC, 1990). Manning's equation forms the basis for the kinematic wave method, as described in detail in Section 4.6. All of the above flood routing methods exist for HEC-HMS and any one of them can be selected in the flood routing setup options. Vflo in Chapter 11 is a good example where KW overland flow combined with storage routing in channels can provide accurate results for most watersheds.

Figure 5–5

(a) Steady-flow water surface profiles. (b) Storage-outflow curve. (This is generic info, created from into in text.)



5.5 APPLICATION OF HEC-HMS TO WATERSHEDS

The HEC Hydrologic Modeling System (version 3.0, 2006; version 3.5, 2010) is a Windows-based hydrologic model that continues to be improved and upgraded over the years. This section contains a brief description of the background, capabilities, and usage of the program; specific information is given on the selection and application of the various methods offered. A detailed applications manual is available from HEC (Ford et al., 2008). Example 5–1 gives the user a hands-on introduction to HEC-HMS, and Section 5.6 contains a case study of a typical watershed area that demonstrates the application of the model in detail. The beginning student is advised to refer to the most recent user manuals for the actual model setup, tutorials, and applications. Several datasets and examples are provided in Appendix E in the text and on the textbook website.

In HEC-HMS, a project consists of three separate parts: the basin model, the meteorologic model, and the control specifications. A background map containing subwatershed boundaries and streams can be entered from a GIS mapfile as a visual reference, but it is not used for any calculations. Mapfiles can be used as a background by selecting “background maps” under the View file menu and entering the name of the existing mapfile. The

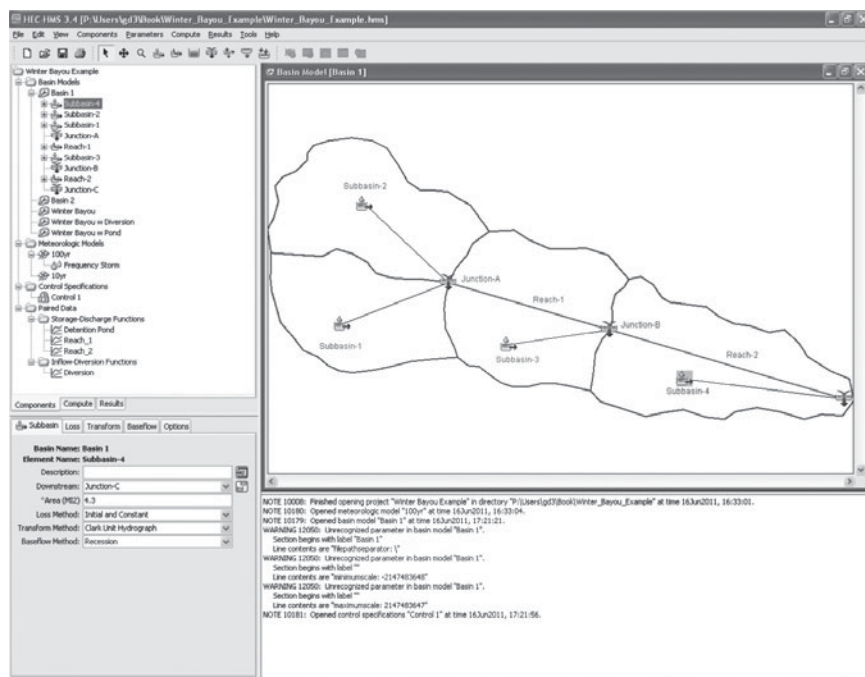


Figure 5–6(a)
Screenshot of a basin model in HEC-HMS version 3.5.

three model sections are accessed by the main screen, within the Watershed Explorer (WE) screen, which is the top window on the left side panel shown in Figure 5–6(a). This screen links to all the data and tools through the menus for the various main components. The user can select the models in the Watershed Explorer, and edit their components in the bottom window on the left side panel.

The Basin Model

The basin model contains the basin and routing parameters of the model as well as connectivity data for the basin. The GUI uses a simple click-and-drag method to place subbasins, reaches, reservoirs, junctions, diversions, sources, and sinks together to form the watershed. Each element can be given a name and description, and the user may select the method of calculation to be used by the model. The basin model is merely a representation of the actual watershed, and the visual location and sizes of each element do not matter as long as the numerical data and connectivity are correct. HEC-HMS then connects some objects placed at either end of routing reaches. Multiple elements can connect downstream to one element, but one element cannot have multiple downstream connections. The user must be careful to connect

objects in the correct direction of flow, which can be checked by selecting the option to show flow directions from the toolbar. A series of icons are used to represent the various elements within the model such as subbasin, routing reach, junction, reservoir, or outlet and are arranged in a horizontal line beneath the main menu. The icon representations for subbasins and junctions can be identified in basin model in Figure 5–6(a).

Subbasins represent the physical areas within the basin and produce a discharge hydrograph at the outlet of their respective areas. The hydrograph produced is calculated from precipitation data minus the losses. The resulting precipitation excess is transformed using a UH methodology to compute runoff at the outlet, which is then added to baseflow. Each component can be calculated using several methods. The area, loss rate, runoff, and baseflow inputs for each subbasin are entered into the basin model for each one using the Watershed Explorer.

Loss rates can be simulated by one of several methods. For event modeling, techniques include initial and constant, SCS curve number, gridded SCS curve number, and Green and Ampt methods. The one-layer deficit and constant model can be used for simple continuous modeling. For modeling of complex infiltration and evapotranspiration environments, the five-layer soil moisture accounting model can be used. The method is changed in a simple menu-driven window, and the input data are entered on the “Loss” tab in the editor (Fig. 5–6(a)).

Transform methods, which convert rainfall excess into surface runoff, can also be simulated using a variety of tools. HEC-HMS includes the popular Clark TC & R and SCS UH techniques, as well as the Snyder Method or user specified UH. Spatially distributed runoff can be computed with the quasi-distributed linear transform of cell-based precipitation and infiltration. The Modified Clark method (ModClark) is a linear quasi-distributed UH method that can be used with gridded precipitation data. If the ModClark transform with gridded rainfall is used, a file that contains characteristics of subbasin grid cells is required. HEC-HMS can handle grid cell depiction of the watershed for distributed runoff computations. The kinematic wave method with multiple planes and channels is also included (see Chapter 4 for detail). Input data are entered in a simple menu-driven window.

Baseflow takes into account normal flow through a channel or the effects of ground water. HEC-HMS offers two methods for baseflow calculation: recession and constant monthly. The recession method is an exponential decay function of a defined starting baseflow. For the constant monthly method, the user simply enters a constant baseflow value for each month. No baseflow is also an option, and in simple hydrologic models over short time periods or highly urbanized basins with channels, baseflow can usually be neglected.

Flood routing in HEC-HMS offers a number of options for the reaches and routing of flood hydrographs. The Muskingum method can be used for general routing; routing with no attenuation can be modeled with the lag

method. The most popular and accurate is the Modified Puls method, which can be used to model a reach with a user-specified storage–outflow relationship. The S-Q relationship is derived from a hydraulics model such as HEC-RAS (Chapter 7).

Channels with trapezoidal, rectangular, triangular, or circular cross sections can be modeled with Modified Puls, the kinematic wave or Muskingum–Cunge method (see Chapter 4). Channels with overbank areas can be modeled with the Muskingum–Cunge eight-point method, which takes coordinates at eight points in a cross section of the channel. The addition of options for subbasins and reaches allows different users to run HMS using the methods most appropriate to their watersheds. Input data are entered in a simple menu-driven window within Watershed Explorer. Homework problems in this chapter expand upon this further.

Reservoirs store the inflow from upstream elements and produces an outflow hydrograph based on a monotonically increasing storage–outflow relationship. A reservoir can be entered with one of three possible types of relationships: storage vs. outflow, elevation vs. storage vs. outflow, or elevation vs. area vs. outflow. The outflow structure must be well understood in order to develop an accurate storage–discharge relation. The inflow entering the reservoir must be contained with the minimum and maximum values of the data entered. The user must also select an initial condition of storage, elevation, outflow, or select inflow equal to outflow. The model assumes that elements have a level pool, such as ponds, lakes, or reservoirs. The effect of adding a detention pond to a basin can be modeled by using a reservoir. The input window for a reservoir is used to relate storage to outflow. A reservoir icon is used to represent storage at any point in the watershed and is then connected downstream to a junction.

Sources are elements that represent a discharge into the basin as an observed hydrograph or a hydrograph generated by a previous simulation. They often are used to represent inflow from reservoirs, unmodeled headwater regions, or a watershed outside the region. This may be entered as gage data or a constant discharge. **Sinks** are elements that have an inflow and no outflow. The only inputs are the name and description of the sink. It may represent the lowest point of the drainage area or the outlet.

Diversions for hydrologic models use a simple table relating inflow to diverted flow and routed flow. These relationships can be determined using geometric calculations and hydraulic models. Diversions will have two “downstream” connections, one being the routed path and the other the diverted path. The user specifies the diverted flow through the use of a table (inflow-diversion), and whatever flow is not diverted will travel the main path. If the “connected” option is selected, then the diverted flow will return to the watershed at the downstream location.

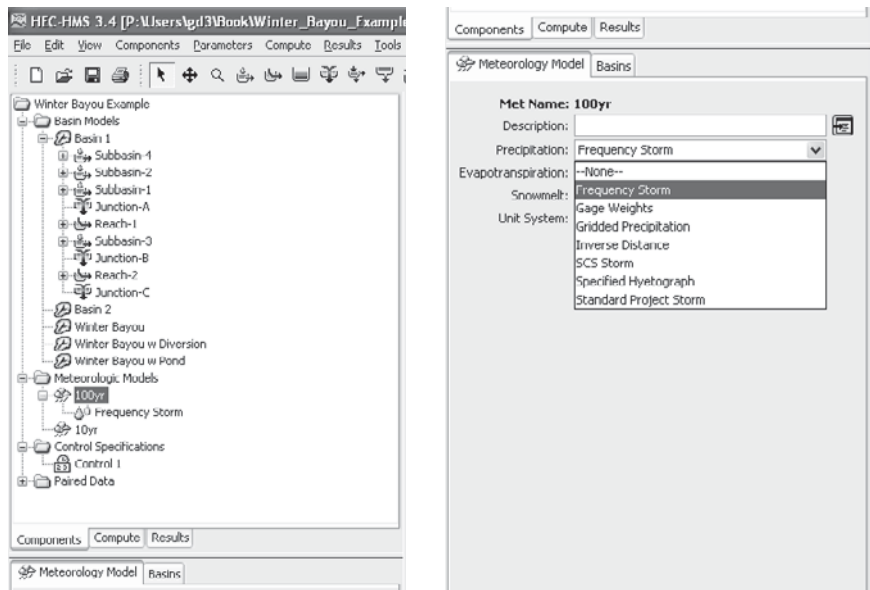
Figure 5–6(a) shows a typical basin model with a mapfile in the background. In very complicated watersheds where icons are often too crowded to display all labels, the user can identify objects by clicking on the buttons

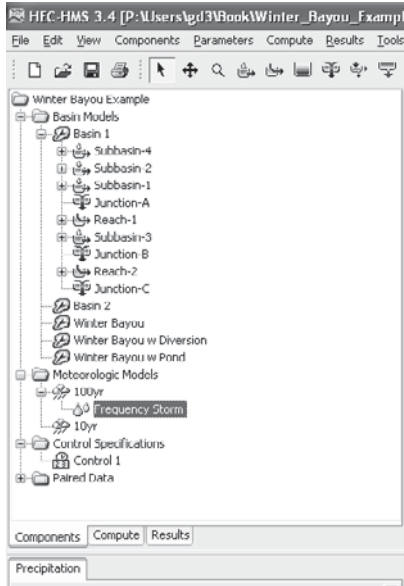
in the toolbar of the basin model. The selected item will be highlighted gray in the basin model. The toolbar also contains a magnifying button to zoom into a selected area, and the user can zoom out of an area by going to “Zoom Out” under View in menu the main menu. Objects can only be selected using the arrow tool.

The Meteorologic Model

The meteorologic model contains the precipitation data, either historical or hypothetical, for the HEC-HMS model. The model contains a number of options for modeling precipitation and can also account for evapotranspiration. Examples of historical precipitation inputs include hyetographs, gage weighting, and inverse-distance gage weighting (Fig. 5–6(b)). The program can handle an unlimited number of recording and nonrecording gages, and gage data can be entered manually, imported from an existing DSS file, or based on an Excel file, as described later. In addition, HEC-HMS has the capability to model gridded rainfall, such as NEXRAD-estimated radar rainfall (Chapters 11 and 12). Hypothetical precipitation data can be used for design storms based on frequency storm analysis and standard project storm (SPS) models. Figure 5–6(c) presents an example of how the 100-year frequency storm for Houston, TX, would be inputted into the model. Note that by clicking on the meteorological model the user can select the type of storm (Fig. 5–6(b)), and by clicking on the type of storm directly beneath the meteorological model the user can edit the specifications for that storm (Fig. 5–6(c)).

Figure 5–6(b)
Meteorologic
model options.





Components Compute Results

Precipitation

Met Name: 100yr

Probability: 1 Percent

Input Type: Annual Duration

Output Type: Annual Duration

Intensity Duration: 15 Minutes

Storm Duration: 1 Day

Intensity Position: 50 Percent

Storm Area (MI2)

5 Minutes (IN)

*15 Minutes (IN) 2.0000

*1 Hour (IN) 4.3200

*2 Hours (IN) 5.8300

*3 Hours (IN) 7.3300

*6 Hours (IN) 9.8200

*12 Hours (IN) 12.310

*1 day (IN) 13.890

2 Days (IN)

4 Days (IN)

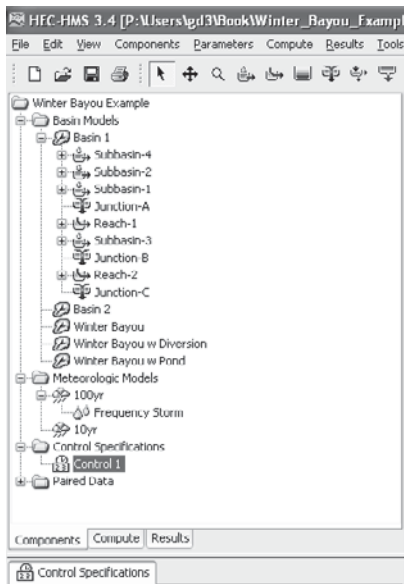
7 Days (IN)

10 Days (IN)

Figure 5–6(c)
Inputting frequency storm information into the meteorologic model.

Control Specifications

The control specifications contains all the timing information for the model, including the start time and date, stop time and date, and computational time step of the simulation (Fig. 5–6(d)). This allows for easy organization of modeling data, which requires a separate dataset describing all aspects of



Components Compute Results

Control Specifications

Name: Control 1

Description:

*Start Date (ddMMYYYY) 20May2011

*Start Time (H:M) 00:00

*End Date (ddMMYYYY) 24May2011

*End Time (H:M) 00:00

Time Interval: 15 Minutes

Figure 5–6(d)
HEC-HMS Control Specifications.

each independent modeling run. A series of runs can be easily organized using this option with several different scenarios. HEC-HMS can also handle continuous hydrograph simulation over long periods. It is standard to run the model several days past the end of the storm to ensure that the falling limb of the computed hydrograph is completely captured.

Data Handling

A major improvement for HEC-HMS is the use of the Data Storage System, or HEC-DSS, used to manage time-series and tabular data. The system was the result of a need in hydrologic engineering to store a variety of types of data in a standardized format. Previously, data formatted for one program would need to be entered into another format for a different program by hand by each user. Each program would then use separate functions to analyze and graph the data. The HEC-DSS software is the result of an effort to make hydrologic data management more efficient and allow for the HEC family of programs to use the same database.

HEC-HMS stores all its output in a DSS file and can store much of its tabular or time-series input data in one or more DSS files. For example, reservoir storage–outflow relationships as well as precipitation and observed flow time-series data are stored in a DSS file. The user either can input the data manually into HEC-HMS using the appropriate dialog boxes or can tell HMS to access the data from a DSS file where they are stored. HEC-HMS can use more than one DSS file for a project. There is always one DSS file associated with the project, which stores output. A second DSS file (or more) can hold precipitation and observed flow data as well as other user-entered data (see user’s manual for more detail).

While the HEC-DSS software package is an improved means of managing hydrologic data, there is no easy way to transfer the data from a spreadsheet to a HEC-DSS file. HEC-DSS utility software available for download from the USACE website given in Appendix E will allow import and export of data from text files for input to HEC-HMS.

Running HEC-HMS and Viewing Results

The user may specify different data sets for each component within a project and then run the hydrologic simulation using different combinations of models. For example, one can run a 10-year or a 100-year frequency storm model using the same basin model and control specifications to compare the resulting flows. Or, one can model the effects of adding diversions and reservoirs to a basin by saving it under a different name, altering the new basin model, and running the simulation under the same precipitation and control specifications. With several basin models saved in the same project, running simulations with various models is a simple task. In order to create the scenario for a particular run, go to “Simulation run” under Compute in the main

file menu. Select one basin model, one meteorologic model, and one control specification. Then run the scenario by selecting “Select run,” and subsequently “Compute run,” both of which are under Compute in the main file menu. If changes are made to one of the models after a run with the same configuration has been calculated, select the correct run to recalculate. Note that the “Compute” tab beneath the Watershed Explorer allows the user to easily select different runs.

Results can be viewed by going to the “Results” tab beneath the Watershed Explorer, clicking on any object (e.g., subbasin or junction) in the basin model, and selecting from the menu. The program gives the resulting times and flows for each basin element and its immediate upstream elements in the form of a graph, summary table, or time-series table. The graph shows time vs. flow, the summary table shows peak flows and corresponding times, and the time-series table gives the inflow and outflow for each time step. For subbasins, HMS also shows the precipitation, loss, excess, and base-flow in each form of viewing results. Junction plots show tributary hydrographs and their combined result. Examples of resulting hydrographs are presented at the end of Section 5.6. Subbasin output hydrographs plotted with rainfall can be obtained by clicking on the appropriate icon. Summary table information is also available for each junction or subarea.

Other Features

HEC-HMS includes advanced features such as parameter estimation with optimization, soil moisture accounting, GIS and grid-cell hydrology, snow-melt simulation, and improved hydraulics. HEC-GeoHMS is a companion program that allows for the creation of HEC-HMS projects from GIS sources, including digital elevation models, land use data, and other electronic sources (see Chapter 10).

The parameter estimation and optimization function is used to compare resulting hydrographs to observed hydrographs, so at least one element in the basin must have observed data. The program automatically estimates the parameters in order to find the best fit of the generated hydrograph to the observed one for one element. It allows the user to set initial values for the parameters of each element along with maximum and minimum values so that the parameter values estimated by the program must fall in a reasonable range. The optimization function measures the degree of variation between the two hydrographs based on four possible choices: peak-weighted RMS error, sum of squared residuals, sum of absolute values, or percent error in peak flow. The program is run iteratively until the user is satisfied with the objective function value, which is zero if the hydrographs match identically.

The development of HEC-HMS is a project that continues to evolve with time; users should periodically check the HEC U.S. Army Corp of Engineer’s (ACOE) website for updates. For further information and help

with HEC-HMS, the user's manual can be downloaded from the HEC web-site: <http://www.hec.usace.army.mil>. The latest updates and other information can be found on the ACOE's web page (see Appendix E for more details). Version 3.5 is a significant improvement over earlier versions and is available to anyone for free (see Appendix E).

EXAMPLE 5-1**SMALL-WATERSHED EXAMPLE (HEC-HMS)**

A small undeveloped watershed has the parameters listed in the accompanying tables. A unit hydrograph and Muskingum routing coefficients are known for subbasin 3, shown in Figure E5-1(a). TC and R values (Clark UH from Table 5.10) for subbasins 1 and 2 and associated SCS curve numbers (CN) are provided as shown. A 5-hr rainfall hyetograph in in./hr is shown in Figure E5-1(b) for a storm event that occurred on May 5, 2011. Assume that the rain fell uniformly over the watershed. Use the information given to develop a HEC-HMS input data set to model this storm. Run the model to determine the predicted outflow at point B .

Subbasin Number	TC (hr)	R (hr)	SCS Curve Number	% Impervious (%)	Area (mi ²)
1	2.5	5.5	66	0	2.5
2	2.8	7.5	58	0	2.7
3	—	—	58	0	3.3

Input UH for	Time (hr)	0	1	2	3	4	5	6	7
Subbasin 3:	U (cfs)	0	200	400	600	450	300	150	0

Muskingum coefficients: $x = 0.15$, $K = 3$ hr, area = 3.3 sq mi.

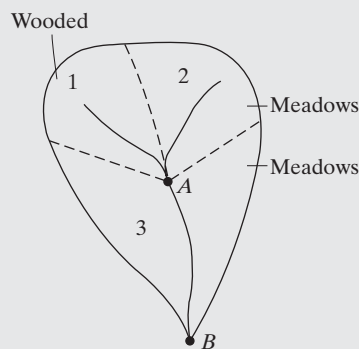


Figure E5-1(a)

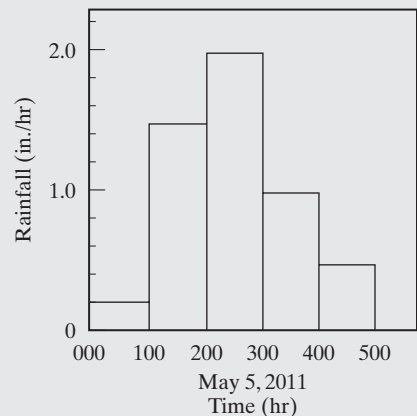


Figure E5-1(b)

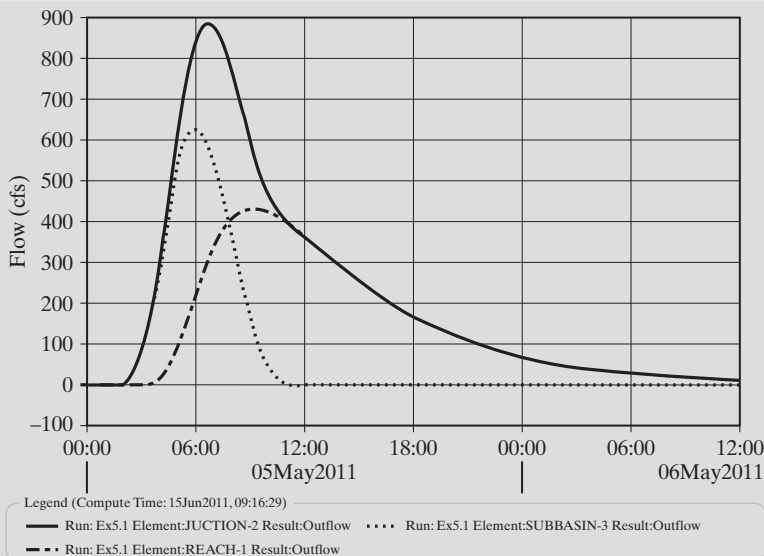


Figure E5-1(c)
Outflow hydrograph at outlet with reach and subbasin inputs.

Table E5-1 lists the general steps for creating a project in HEC-HMS and the corresponding steps for running the watershed from Example 5-1. Run the project in HEC-HMS, and evaluate the final outflow hydrograph (Fig. E5-1(c)).

SOLUTION

Table E5-1 Steps for Example 5-1

- 1 Open HEC-HMS, start a **New Project** under the **File** menu, and name the project *Example 5-1*.
- 2 Select the **U.S. Customary** Units.
- 3 Set up the subbasin model by selecting the **Basin Model Manager** from the **Components** menu, clicking **New...**, and naming it *Basin 1*.
- 4 Click on *Basin 1* from the drop down folder *Basin Models*. These are located in the **Watershed Explorer Window**.
- 5 From the HEC-HMS toolbar, select the **Subbasin Creation Tool**. Click anywhere in the **Basin Model** window to create a new subbasin. Name the new feature *Subbasin 1*, and click the **Create** button. Repeat this step two more times for *Subbasin 2* and *Subbasin 3* so that they are arranged according to Figure E5-1(a).
- 6 Click on the **Junction Creation Tool** in the HEC-HMS toolbar and click below Subbasins 1 and 2. Name this *Junction A* and click **Create**. Repeat this step for *Junction B*, and place it below Subbasin 3.
- 7 Select the **Reach Creation Tool** from the toolbar and draw a reach between Junction A and Junction B. Name this reach **Main** and click the **Create** button. Check that it is drawn correctly by dragging the Junctions to make sure the Reach is attached.
- 8 For each Subbasin, make sure that the corresponding Junctions are in the **Downstream** option. (Junction A for Subbasins 1 and 2 and Junction B for Subbasin 3).

(continued)

Table E5-1 (Continued)

- 9 Click on any of the subbasins in your model and look at the **Subbasin** tab in the data window. Define the **Area** for each subbasin as defined in the first table. Under the **Loss Method**, select **SCS Curve Number** for each subbasin. The **Transform Method** will be **Clark Unit Hydrograph** for Subbasins 1 and 2, but will be **User-Specified Unit Hydrograph** for Subbasin 3. Select **None** as the **Baseflow Method** for each of the subbasins.
- 10 On the **Loss** and **Transform** tabs in the Component Editor window, input the appropriate values given in the first table for each subbasin.
- 11 Select **Components** from the main menu and pick **Paired Data Manager**. Select **Unit Hydrograph Curves** from the **Data Type** pull-down and click the **New...** button to initiate a new paired data set. Name this set *Subbasin3UH*, click the **Create** button, and close the Paired Data Manager window.
- 12 In the Watershed Explorer window, expand the **Paired Data** and **Unit Hydrograph Curves** and select the **Subbasin3UH** data set. Change the time interval value to *1 hour*. The **Data Source** will be *Manual Entry*, so click the **Table** tab to manually enter the unit hydrograph for Subbasin 3. Enter the data from the second table.
- 13 Now that the paired dataset is created, click on the **Subbasin 3** element and under the **Transform** tab, select *Subbasin3UH*.
- 14 Click on the **Main** reach and set its **Routing Method** to **Muskingum**. In the Component Editor window, select the **Routing** tab and enter given **x** and **K** coefficients (use a subreaches value of 6).
- 15 To add precipitation data, select **Components** from the main menu and click **Meteorologic Model Manager**. Select the **New...** button and name the storm event *5May11*. Click the **Create** button and close the Meteorologic Model Manager.
- 16 In the Watershed Explorer window, expand **Meteorologic Models** and select the **5May11** event. In the Component Editor window, select *Specified Hyetograph* for **Precipitation** on the Meteorology Model tab. Next, select the **Basins** tab and select YES under the **Include Subbasins** question.
- 17 Next we define a gage location where this storm event was observed. Under **Components** in the main menu, select **Time-Series Data Manager** and confirm that **Data Type** is set to *Precipitation Gages*. Click the **New...** button, name this *Gage 1*, click the **Create** button and close the Time-Series Data Manager.
- 18 Expand the **Time-Series Data**, **Precipitation Gages**, and the **Gage 1** items in the Watershed Explorer window. In the Component Editor window, change the Time Interval to *1 hour*. Clicking on the Time Window tab will allow you to modify the time domain for this event. Change storm date and time to 00:00 to 5:00 on 5May2011. Click the **Table** tab to manually input the incremental rainfall values for this storm (Fig. E5-1(b)).
- 19 Next, attach this gage data to the defined hyetograph by selecting *Specified Hyetograph* under 19Jun10 in the Watershed Explorer window. Select *Gage 1* for each of the subbasins shown in the Component Editor window.
- 20 Select **Components** in the main menu and click on **Control Specifications Manager**. Click on the **New...** button, add a control specification named *Test01*. Click the **Create** button and close the Control Specifications Manager. Expand the new Control Specifications item in the Watershed Explorer window and select the *Test01* item. Define the Start Date/Time and End Date/Time as 5May2011 at 00:00 and 6May2011 at 12:00. Select *10 minutes* for the time interval.
- 21 Now we can finally define a project simulation. Select **Compute** from the main menu and click on **Create Simulation Run**. Name the run *5May11* and click the "Next" button. In the next dialogs, select **Basin 1** for the basin model, **5May11** for the meteorologic model, and **Test01** for the control specifications.
- 22 To run your model, select **Compute** in the main menu, and choose **Select Run** and **5May11**. Next, select **Compute** (main menu) and *Compute Run [5May2011]*.
- 23 Click the **Results** tab in the Watershed Explorer window, expand the Simulation Runs item and click on any of the elements to view the simulation results.

Watershed Description

This section analyzes streamflow in a hypothetical watershed, Winterday Creek (Fig. 5–7(a)), and the separate effects of adding a detention pond and a diversion. The watershed, with a total area of about 10.99 mi², is made up of four subbasins with varying sets of hydrologic parameters. In their existing condition, the subbasins in this watershed range from 18% imperviousness (subbasin 2) to 60% imperviousness (subbasin 4). The subbasins are relatively flat (10 to 20 ft/mi) and range in size from 2.0 to 4.3 mi² (see Table 5.8(a)).

Physical Parameters

The physical parameters for each of the subbasins in the hypothetical watershed are shown in Tables 5–8(a,b). Theoretically, parameters of main interest such as channel slopes, channel lengths, percent development, and percent conveyance (the ratio of flow in the channel to total flow) could be determined for any watershed, given the existence of topographic maps and land use data. Chapter 10 presents alternative elec-

5.6

HEC-HMS WATERSHED ANALYSIS: CASE STUDY

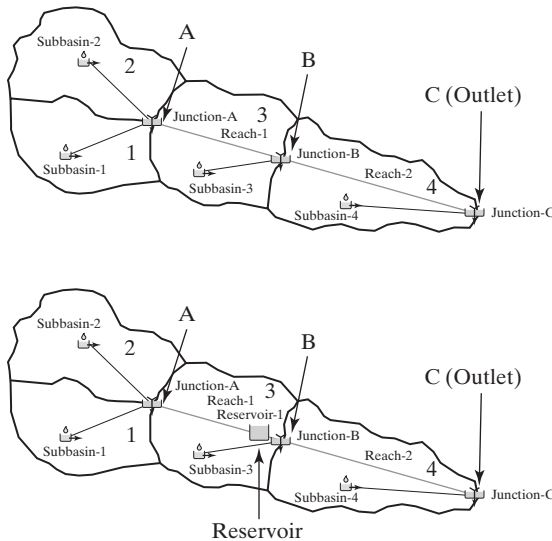


Figure 5–7(a)
Watershed map
showing subarea
delineation of
Winterday Creek.

Figure 5–7(b)
Watershed map
showing the loca-
tion of an added
detention pond.

Table 5–8(a) Physical Parameters for the Hypothetical Watershed

Subbasin	Area (mi ²)	L (ft)	L _{ca} (ft)	S (ft/mi)	Development (%)	Conveyance (%)
1	2.0	7000	4000	20	30	80
2	2.32	9000	5050	10	28	80
3	2.37	8750	4975	10	75	100
4	4.3	10,000	6340	10	80	100

Table 5–8(b) Infiltration Loss Data for Each Subbasin

Subbasin	Impervious (%)	SCS Curve Number
1	20	78
2	18	80
3	52	85
4	60	90

tronic data available in GIS formats and DEMs (digital elevation models) for use in hydrologic watershed analysis. Detailed examples are provided in Chapters 10 and 11.

Rainfall Data

The HEC-HMS model can accept rainfall data in a number of ways: historical data from recording or non-recording gages or design storm data. When using data from gages spaced unevenly across a given watershed, the gage values should be weighted accordingly. The Thiessen method is the usual approach. Although not discussed in this example (see Chapter 1), problems 5.10–5.11 address the Thiessen method for subbasin data when only gage values are known.

The 10-yr and 100-yr synthetic 24-hr design storms for the Houston area are depicted in Table 5–9 and Figure 5–8 (TSARP, 2002). For example, the HEC-HMS model accepts input data for the 15-min, 2-hr, 3-hr, 6-hr, 12-hr, and 24-hr total design rainfalls, and then distributes the data as a bar graph by centering the maximum at a user-specified point in time (i.e., 33%, 50%, 67%, or 75% of storm duration), according to agency standards. Thus, one can input any design rainfall into HEC-HMS, calculate the outflow hydrograph, and compare the result with a given historical rainfall. In this way, one can determine the approximate frequency of occurrence of the historical storm. In practice, a range of design storms including the 2-yr, 5-yr, 10-yr, 25-yr, 50-yr, 100-yr, and 500-yr rainfalls are evaluated in most flood studies.

Table 5–9 Houston Frequency Rainfall, Updated 2002 (TSARP)

Duration	Houston Frequency Rainfall				
	5 yr	10 yr	25 yr	50 yr	100 yr
15 min	0.59	0.69	0.83	0.95	1.08
1 hr	2.37	2.76	3.33	3.8	4.32
2 hr	2.96	3.51	4.32	5.02	5.83
3 hr	3.55	4.25	5.3	6.24	7.33
6 hr	4.31	5.27	6.76	8.14	9.82
12 hr	5.14	6.37	8.31	10.12	12.31
24 hr	5.95	7.35	9.51	11.50	13.89

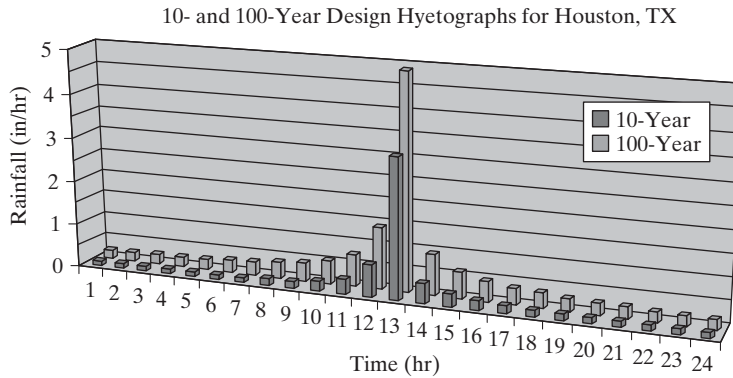


Figure 5-8
Design hyetograph
for Houston, Texas.

Unit Hydrograph Data

In the case study of Winterday Creek, the Clark (TC & R) method will be used to illustrate its applications to an urban watershed. The TC & R method is summarized in Table 5-10 with empirical equations that have been developed for the Harris County (Houston) area in Texas. The equations used here are based on the analysis of a number of watersheds along the Texas Gulf Coast (Harris County Flood Control District, 1983; TSARP, 2002). Many similar equations have been generated for other urban areas and regions across the country, and the user of HEC-HMS is strongly urged to evaluate the TC & R method carefully before applying it to any ungaged watersheds.

The physical parameters presented in Table 5-8 were used to develop the values for TC & R shown in Table 5-11. Generally, the lower the percent development and percent conveyance in a subbasin, the higher the value of TC & R . Thus, subbasin 2 has the highest value of TC & R and has the flat-test UH response. Individual values for each subbasin are input into HEC-HMS using the transform tab in the subbasin editor. Note also that SCS Curve Number method was selected for infiltration losses and the corresponding parameters were inserted in the subbasin editor under the loss rate tab (Table 5-8(b)).

Hydrograph and Channel Routing

The HEC-HMS model calculates outflow hydrographs for each subbasin based on unit hydrographs. For this watershed, the flood hydrographs from subbasins 1 and 2 are computed for the outlet of those subbasins (point A in Fig. 5-7(a)). The storm hydrograph from subbasin 3 is assumed to occur at point B , and the hydrograph from subbasin 4 is assumed to occur at point C . It is therefore required that the combined hydrograph from subbasins 1 and 2 at point A be flood routed through reach AB and then combined with the

Table 5-10 Equations for TC & R Method for Houston, Texas

The method requires that $(TC + R)$ and TC are computed from the following formulas, then R is found by subtraction of the two, where $R = (TC + R) - TC$.

$$TC + R = C \left(\frac{L}{\sqrt{S}} \right)^{0.706},$$

where

TC = time of concentration (hr),

R = routing constant (hr),

L = length of channel (outflow to basin boundary) (mi),

S = channel slope (ft/mi),

$$C = 4295[\% \text{ Dev}]^{-0.678}[\% \text{ Conv}]^{-0.967},$$

$$C = 7.25 \quad \text{if } \% \text{ Dev} \leq 18,$$

$$TC = C' \left(\frac{L_{ca}}{\sqrt{S}} \right)^{1.06},$$

where C' is taken from the following for overland slope and % Dev:

S_0 (ft/mi)	% Dev	C'
>40	0	5.12
$20 < S_0 \leq 40$	0	3.79
≤ 20	0	2.46
>40	100	1.95
≤ 20	100	0.94

(If the percent development is between 0 and 100, C' is found by linear interpolation.)

L_{ca} = length along channel to centroid of area (mi),

S_0 = representative overland slope (ft/mi),

% Dev = percent of area that is developed (%),

% Conv = % conveyance, the ratio of flow in channel to total flow (%),

TC = time of concentration (hr),

R = routing constant (hr).

Table 5-11 Computed TC & R Coefficients for Each Subbasin

Subbasin Number	TC & R Coefficients		
	TC	R	Impervious (%)
1	0.3	2.31	20
2	0.57	3.61	18
3	0.36	1.33	52
4	0.44	1.34	60

hydrograph from subbasin 3 at point *B*. Likewise, the combined hydrograph at point *B* is routed to point *C* and then combined with the hydrograph from subbasin 4 to produce the outlet hydrograph. Larger watersheds with more complicated channel networks can be represented in the HEC-HMS model in a similar fashion.

Storage routing data are sometimes difficult to obtain, especially if hydraulic calculations have not been completed for the main channel. Often in well-studied basins, the HEC-RAS model has been applied to develop water surface profiles and hydraulic information for the main channel (refer to Chapter 7). Computations with HEC-RAS or a similar model allow the development of storage–discharge relations for the reaches in the channels, which is then used in HEC-HMS to route the hydrographs from one point to another in the main channel.

Routing information can be entered into HEC-HMS by clicking on each reach in the Watershed Explorer and changing the “Routing Method” in the drop-down menu located in the bottom window on the left side panel. Parameters for the selected routing method are then entered on the “Routing” tab. If the **Modified Puls** method is selected, a storage–discharge relationship must be added by going to the Paired Data Manager under Components in the main menu. The user then specifies the specific storage–discharge relationship for the reach in the “Stor-Dis Function” drop-down menu under the “Routing” tab of the reach. Initial stage or flow can also be specified, but is usually set to zero. The Modified Puls routing method is used when modeling flow in Winterday Creek. Table 5–12 presents storage in ac-ft vs. discharge in cfs for the two reaches in the watershed.

A second method of flood routing that is sometimes used is the **kinematic wave method**. This method works well in areas with a significant overland slope in which the flood wave is essentially translated. The Houston area is exceptionally flat with very little slope, which results in much more attenuation of the flood wave than the kinematic wave method allows for. For this reason, the Modified Puls method is usually a better choice. More

Table 5–12 Channel Storage Routing Data

Reach A-B		Reach B-C	
S ac-ft	Q cfs	S ac-ft	Q cfs
0.0	0.0	0	0
120	120	72.3	465
189	256	114	930
245	315	180.8	1860
276.1	412	325.7	2790
1042.7	1564	842	3720
1800	1901	1664.9	4650
3077	2099	2653.7	5580
4555.8	2200	4379.9	6975

discussion is presented in Chapter 11 for distributed models using kinematic wave routing. The various methods of calculation that HEC-HMS uses are established and have been proven to work well in the field. In reality, however, certain methods are designed to work better with watersheds of certain physical characteristics than with others. It is always important to check that one is using the best method for a given watershed to ensure that the most accurate results are obtained. Figure 5–5 shows a graph to demonstrate the Modified Puls method.

Resulting Hydrographs

The HEC-HMS model was run using data presented in the tables above and using the Clark (*TC & R*) method. Two different rainfalls were tested: the 10-yr frequency design storm and the 100-yr frequency design storm. The rainfall input datasets used for the 10-yr and 100-yr design storms are shown in Table 5–9 and Figure 5–8. Resulting peak outflows for both storms are listed in Table 5–13. In general, it is always useful to check three parameters against the measured gage data in calibrating HEC-HMS input: peak flow, volume under the hydrograph, and time to peak. Some adjustment of watershed parameters is usually necessary to calibrate the model fully.

Once the HEC-HMS input data are calibrated for several measured storm events, predictions for the 10-yr, 100-yr, or any other design storm can be made by changing rainfall input in the model (see Problems 5.6–5.9). The resulting 10-yr and 100-yr design storm hydrographs then provide an estimate of peak flows at various points along the main channel. These flows can be used with cross-section data and hydraulic computations to derive elevations for the 10-yr and 100-yr floodplain using HEC-RAS as described in Chapter 7.

As with any modeling exercise, sources of error in the calibration of HEC-HMS to a watershed include the following:

- 1. Spatially variable input rainfall with too few rain gages across the watershed
- 2. Infiltration or loss inaccuracies due to variable rainfall or changing land use
- 3. Inaccurate estimates of routing coefficients for channels
- 4. Nonlinear hydrographs that cannot be properly represented by linear unit hydrograph theory

Table 5–13 Comparison of Peak Flows for the Hypothetical Watershed

Frequency Design Storm	Peak Flow (cfs)	
	Junction-B	Junction-C Outlet
10-yr	3052	6830
100-yr	4197	9024

Flood Control Alternatives

Once a watershed has been evaluated under existing conditions of land use and channel morphology, the HEC-HMS model can be used to test flood control plans that might include a reservoir or detention pond to increase storage in the watershed. Some diversion of part of the flow to another watershed where storage is available might also be possible. These possibilities are investigated with HEC-HMS in this section.

A powerful feature of the HEC-HMS model is its ability to represent changes in watershed physiography or land use that may occur as a watershed develops. For Winterday Creek, we will consider two examples: (1) a large detention pond at point *B* for the storage of flood waters from subbasins 1, 2, and 3, and (2) the diversion of a portion of the flow at point *B* into a diversion channel that leads to another watershed. In both cases, the watershed will be assumed to have existing land use (Fig. 5–7(b)).

To input a detention pond into HEC-HMS, the storage routing data must be changed to represent the storage–discharge relation for the particular detention pond. The pond data are located in the input through logical placement so that all contributing hydrographs route through the pond. The starting water elevation in the pond depends on the operation of the system. The discharge characteristics can be considered in detail through options available in the model, but generally pipe flow is assumed for low flows and weir flow for flood levels. Details on equations that govern such flows are provided in Chapters 4 and 7.

The input data of the detention pond in HEC-HMS are depicted in Table 5–14. The storage–discharge data are based on pond geometry, which is a function of available land area and available outfall depth. The existing 100-yr peak flow in the water is 4197 cfs at point *B* and 9024 cfs at the outlet (*C*). To control the peak flows at the point *B* and at the outlet of the watershed, a pond was located just upstream of point *B* along the stream (Fig. 5–7(b)). Table 5–15 shows the relative effects of this pond compared with existing conditions. Adding a pond yields a 50% reduction in peak flow at the 100-yr level through the pond and a 24% reduction in peak flow through point *C* (Fig. 5–9).

Table 5–14 Storage-Discharge Relation for the Detention Pond

S (ac-ft)	0	200	450	689	798	950	1200	1800
Q (cfs)	0	146	300	420	501	778	1500	6000

Table 5–15 Comparison of Peak Flows with and without a Detention Pond in the Hypothetical Watershed for the 100-yr Storm

Conditions	Peak Outflow (cfs)		
	Junction-B	Pond	Junction-C
Without pond	4197	—	9024
With pond	2099	2099	6856

Figure 5–9(a)
Outlet hydrograph
for Winterday Creek
modeled under
100-yr storm fre-
quency conditions.

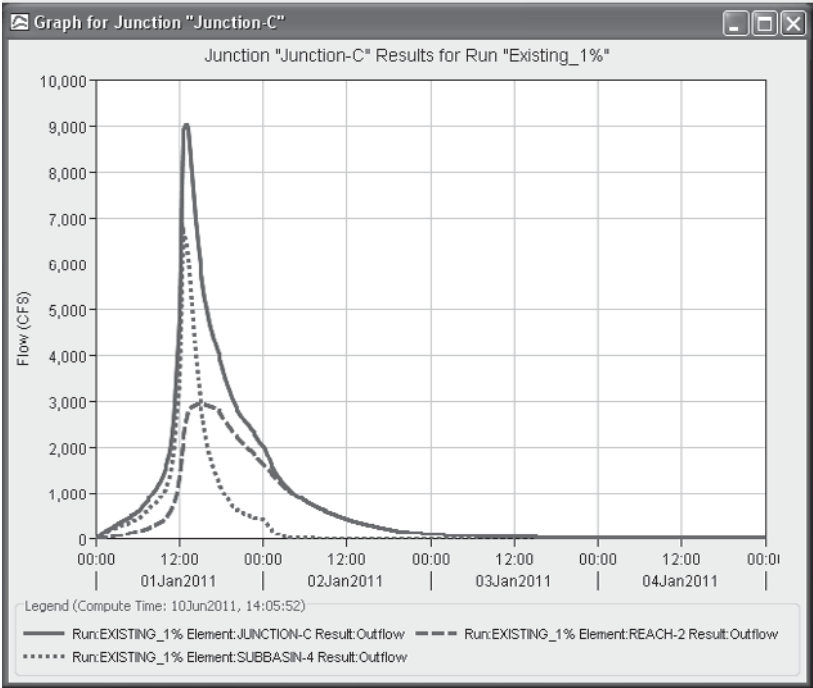


Figure 5–9(b)
Effect of the pond
at point B.

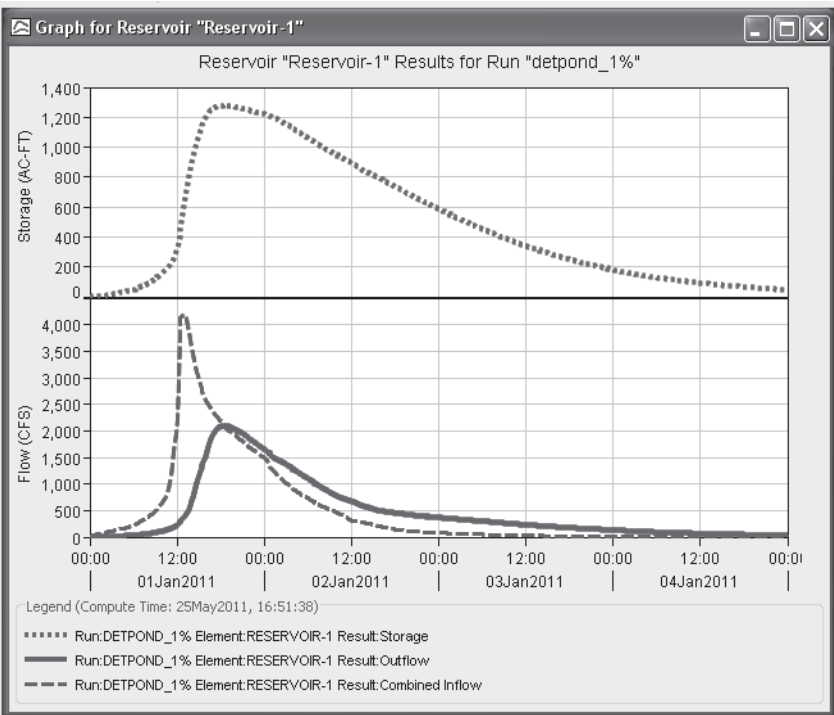


Table 5-16(a) Inflow-Diversion Relation
for the Added Diversion

Inflow (cfs)	Diversion (cfs)
0	0
500	375
1000	750
2500	1545
5000	3150
10,000	5000

Table 5-16(b) Comparison of Peak Flows with and without a Diversion in the Hypothetical Watershed for the 100-yr Storm

Conditions	Peak Outflow (cfs)	
	Junction-B	Junction-C
Without diversion	4197	9024
With diversion	1562	7429

A second alternative for flood control is **diversion** of flood runoff to another watershed. This is accomplished in HEC-HMS via the diversion icon in the basin model, which allows the user to define a flow level above which a certain percentage will be diverted from the main channel (Table 5-16(a)). While this option implies that there must be an adjacent watershed or storage area that can receive the excess flood water, it can be a viable alternative in some cases. For this example, water will be diverted at point *B* and the effect on the resulting hydrograph will be observed just downstream and at the watershed outlet (Table 5-16(b) and Fig. 5-10).

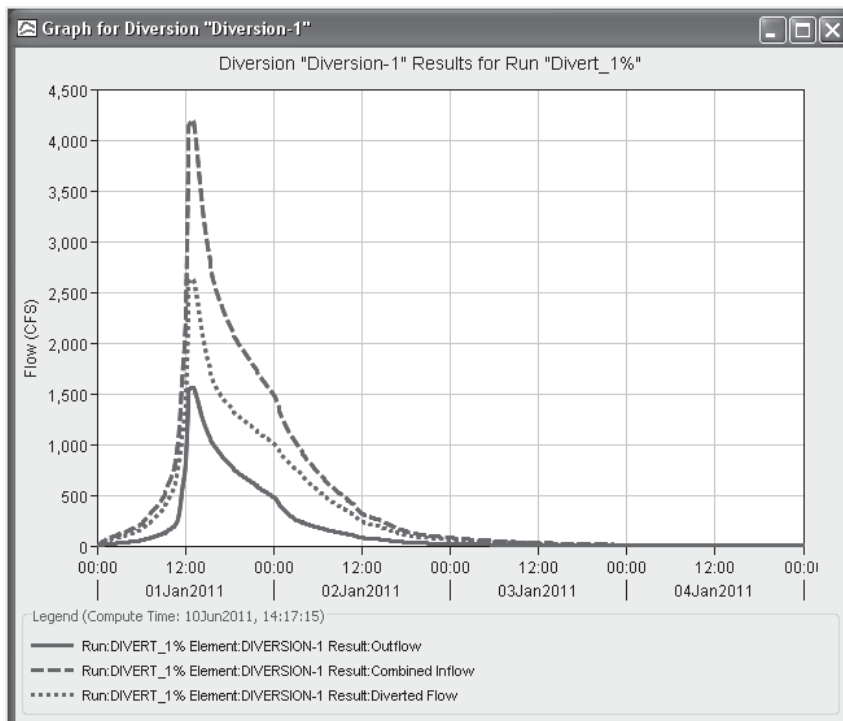


Figure 5-10
Effect of the
diversion at point B.

One final exercise with the HEC-HMS model application to this watershed is a consideration of full development conditions in the watershed, which is consistent with the future conditions in Houston. As the watershed proceeds to full urbanization, significant changes in UH and loss rate parameters will be needed in the model. There would also be associated channelization of laterals and the main stream, which may require changes in the routing parameters in the model. From the previous discussion of effects of urbanization, it should be clear that full development will worsen the flood problem, especially downstream. Homework problem 5.5 explores this issue in more detail. Once developed, the HEC-HMS model for a watershed can be used to evaluate a number of scenarios for flood management and control.

SUMMARY

Simulation models in hydrology have been developed and applied over the last four decades with remarkable success. These models incorporate various equations to describe hydrologic processes in space and time. Complex rainfall patterns affecting large watersheds can be simulated, and computer models can be used to test various design and control schemes. A selected number of the most popular rainfall–runoff models for hydrologic simulation are contained in Table 5–4. Models of some note include HEC-HMS, HEC-RAS, HSPF, SWMM, and Vflo and are some of the most popular and comprehensive models for detailed watershed analysis.

The second half of the chapter is devoted to a detailed presentation of HEC-HMS developed by the U.S. Army Corps of Engineers, considered by many to be the most versatile and most often used hydrologic model. It is the model of choice for most floodplain computations performed in the United States by various agencies such as FEMA and local drainage districts (HEC, 2006, 2010). Homework problems 5.1–5.12 concern a detailed case study and application of HEC-HMS to an urban watershed in Houston, Texas. Input data for the model are developed, using the Clark UH methods (TC & R), Muskingum Routing method, rain gage data, and several design storms, and the results from each are compared. The problems also introduce the effects of development within a watershed and explore the Green and Ampt method to account for infiltration losses. Several flood control alternatives are modeled, including flow diversion and detention ponds at critical locations, to demonstrate the overall capabilities of the HEC-HMS program.

PROBLEMS

Problems 5.1 through 5.12 deal with a hypothetical watershed with the following parameters. Problems 5.1 through 5.5 involve input data setup only, while Problems 5.6 through 5.12 require actual HEC-HMS program computations.

Subbasin Number	Area (ac)	L (ft)	L_{ca} (ft)	S (ft/mi)	Development (%)	Conveyance (%)
1	1280	7000	4000	20	30	80
2	1485	9000	5050	20	28	80
3	2045	8750	4975	10	70	100
4	1517	8230	4115	10	70	100
5	2752	10,000	6340	10	75	100

Assume $S = S_0$ in each subbasin. The infiltration data are as follows:

Subbasin	Impervious (%)	SCS Curve Number
1	20	78
2	18	80
3	52	85
4	55	85
5	60	90

- 5.1** Calculate the TC & R coefficients for each subbasin in the given watershed shown in Figure P5–1. Assume that the equations listed in Table 5–10 are applicable and that $S = S_0$.

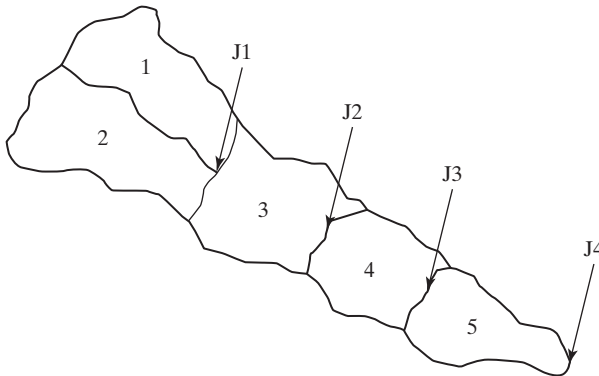


Figure P5–1

- 5.2** Create the basin model of the watershed in HEC-HMS using the data provided. Set up input data for the watershed using Clark's UH method (TC & R). Use Muskingum routing methods and assume that $K = 0.60$ hr and $x = 0.5$ in reaches 1, 2, and 3. Assume two subreaches in each reach and assume baseflow is zero.
- 5.3** Using the data shown in Table 5–9, create two meteorologic models in HEC-HMS using the 24-hr hypothetical design storms for the 100-yr frequency and the 10-yr frequency events. Place the peak center at 50% for both storms and use 20-min maximum intensity duration.
- 5.4** Create a new basin model for HEC-HMS Problem 5.2 to reflect a detention pond near Junction 2. Assume that the pond can be modeled in such a way

that only the runoff from subbasin 3 should be routed through it. The storage–discharge relation is given as follows:

S (ac-ft)	0	200	375	508	860	1123	1500	2120
Q (cfs)	0	146	222	352	480	600	710	1733

- 5.5 The watershed is predicted to grow in the next 20 years and approach 100% development over most of the watershed. If this occurs, certain parameters will change, as shown in the following table. Compute the new TC & R coefficients, and change the basin model of Problem 5.3 to reflect these fully developed conditions. Assume that the Muskingum values used in Problem 5.2 will still be correct.

Subbasin	Development (%)	Conveyance (%)	Impervious (%)	SCS Curve Number
1	50	100	38	83
2	75	100	30	81
3	100	100	80	94
4	100	100	85	94
5	100	100	85	94

- 5.6 Run the HEC-HMS model with $\Delta t = 20$ min to reflect the existing conditions, the effect of the pond with existing conditions, fully developed conditions, and the effect of the pond with fully developed conditions for both the 100-yr and the 10-yr design storms. Compare the peak flows at Junction 3 for the various run combinations.
- 5.7 Set up a diversion at Junction 2 of the existing condition basin so that Reach 1 and subbasin 3 flow into it. Use the inflow–diversion relationship from Table 5–16, and compare the peak outflows at Junction 3 of the new basin model for the 10-yr and 100-yr storm events.
- 5.8 Use the Existing Basin conditions from Problem 5.2, and add a detention pond in the two largest subbasins, using the storage–discharge relationship from Problem 5–4 for each. Run the 10-yr and 100-yr storm events, and compare the hydrograph and peak outflow at the outlet to that of the existing basin model. Discuss the differences.
- 5.9 Use the Existing Basin conditions to make a new model, setting the loss method to Green and Ampt for each subbasin. Enter the Green and Ampt parameters shown in the following table (values come from Table 2–4) and run the 10-yr and 100-yr storm event. Compare the resulting hydrographs and peak flows at Junctions 1 and 4.

Subbasin	Initial Content (%)	Saturated Content (%)	Suction (in)	Conductivity (in/hr)
1	0.232	0.464	8.22	0.039
2	0.232	0.464	8.22	0.039
3	0.232	0.464	8.22	0.039
4	0.232	0.464	8.22	0.039
5	0.232	0.464	8.22	0.039

- 5.10** Rain gage data are given in Table P5–10(a) for three rain gages within the watershed. Weight the gage data over the five subareas according to the weighting values given in Table P5–10(b) (see Example 1–3 in Chapter 1). Set up the meteorological model for subareas 1 through 5 in the watershed to reference this storm's rainfall data. For example, subbasin 2 receives 1/3 of gage 100R rainfall and 2/3 of gage 200R rainfall. Enter the actual rain gage data and weights into HEC-HMS.

Table P5–10(a) Cumulative Rainfall Data

Time (hr)	Gage Numbers		
	100R	200R	300R
0	0.0	0.0	0.0
1	0.60	0.50	0.30
2	1.03	1.10	0.93
3	1.06	1.10	0.95
4	1.29	1.20	1.05
5	1.94	1.40	1.13
6	4.18	2.50	1.38
7	4.32	2.90	1.88
8	4.38	3.00	1.96
9	4.54	3.10	2.25
10	4.70	3.20	2.36
11	4.76	3.40	2.58
12	4.84	3.50	2.65
13	4.88	3.50	2.69
14	4.90	3.50	2.71

Table P5–10(b) Precipitation Gage Weights

Subbasin	Gage Numbers		
	100R	200R	300R
1	1.00	0.00	0.00
2	0.33	0.67	0.00
3	0.00	0.61	0.39
4	0.00	0.00	1.00
5	0.00	0.00	1.00

- 5.11** Run the given storm through the watershed using the Existing Basin conditions. Find the peak flows for this storm at Junction 2 and at the outlet.
- 5.12** For the Muskingum coefficients given in Problem 5.2, determine the dimensions of a wide rectangular channel needed to carry the 100-yr flow under existing development conditions. Use a Manning's n value of 0.04, and assume a channel $L = 10,000$ ft and $S = 0.001$. (*Hint:* Use Muskingum's K equation and Manning's equation.)

Problems 5.13 – 5.15 refer back to Example 5–1 in the Chapter.

- 5.13** Rework Example 5–1, assuming the same type of soil and that the woods in subbasin 1 have grown much denser and provide good cover. Run the revised HEC-HMS and then graph the original and the updated discharge hydrographs from subbasin 1. How does the curve number affect the amount of runoff?
- 5.14** Rework Example 5–1 as described in Problem 5.13 and also assume that subbasin 2 has been developed into 1/4-ac residences. Let the values for $TC = 1.14$ and $R = 3.5$. Plot the final hydrograph for the whole basin vs. the original one in Example 5–1. Discuss the differences.
- 5.15** Alter the rainfall in Example 5–1 by using the following hourly cumulative rainfall measurements:

Time (hr)	0	1	2	3	4	5	6	7	8	9	10
Rainfall (in.)	0	0.47	0.74	1.34	2.64	2.87	3.08	3.95	5.16	5.44	5.44

Assume that there is a detention pond at point *B* of the basin that is used for flood control. Assume a storage–discharge relationship $S = k \times Q$, where k is equivalent to 0.75 ac-ft/cfs. Prepare a graph of the outflow from the basin, with and without detention, and compare results.

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Chapter 6

Urban Hydrology



The Chicago River which serves as a link between the Great Lakes and Mississippi River Valley. (Photo courtesy Courtney Ray.)

Scope of This Chapter

This chapter describes techniques commonly applied in urban hydrology, emphasizing techniques not already discussed earlier in the text and modifying conventional techniques specifically for the urban situation. Modifications include unique loss estimates for calculation of rainfall excess, decreased lag times in unit hydrograph (UH) procedures, and emphasis on kinematic wave techniques for overland flow routing. The choice of input rainfall for construction of a design hyetograph is of special importance in urban hydrology because impervious urban surfaces rapidly convert the rainfall to runoff, such that short-time increment variations in the rainfall hyetograph usually create similar variations in the runoff hydrograph. Sewer hydraulics and options for control of urban flooding are discussed.

Several comprehensive computer models also are available for application in urban situations; the chapter culminates in a review of such models plus a case study involving the EPA Storm Water Management Model, SWMM5. A considerable volume of literature is available on urban hydrology, including

6.1 CHARACTERISTICS OF URBAN HYDROLOGY

texts by Moffa (1990), Urbonas and Stahre (1993), Nix (1994), Field et al. (2000), Mays (2001b), Debo and Reese (2003), and Field and Sullivan (2003) and design guidance that includes Urbonas and Roesner (1993) and Water Environment Federation and American Society of Civil Engineers (1992, 1998). In addition, numerous design manuals are available from state departments of transportation and other public agencies, such as King County (1995) and Denver Urban Drainage and Flood Control District (see Appendix E). Many agencies make these manuals freely available on their websites.

Introduction

Although the same physical principles hold as elsewhere in the hydrologic cycle, the hydrology of urban areas is dominated by two distinct characteristics: (1) the preponderance of impervious surfaces (e.g., pavement, roofs) and (2) the presence of man-made or hydraulically “improved” drainage systems (e.g., a sewer system). Thus, the response of an urban catchment to rainfall is much faster than that of a rural catchment of equivalent area, slope, and soils. In addition, the runoff volume from an urban catchment is larger because there is less pervious area available for infiltration. These well-documented characteristics are illustrated in Figure 6–1. While the

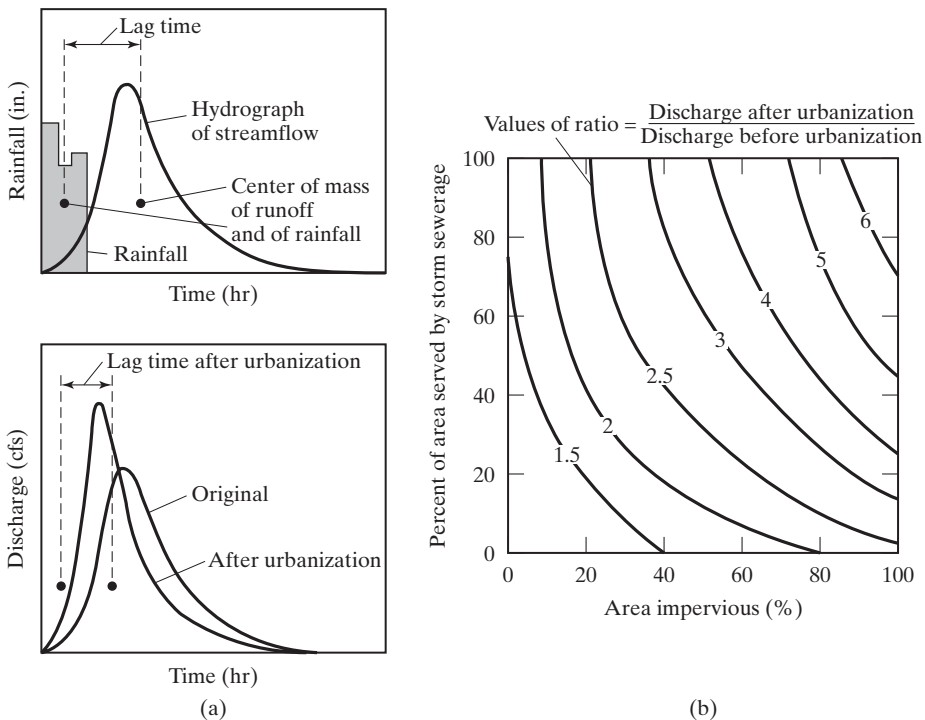
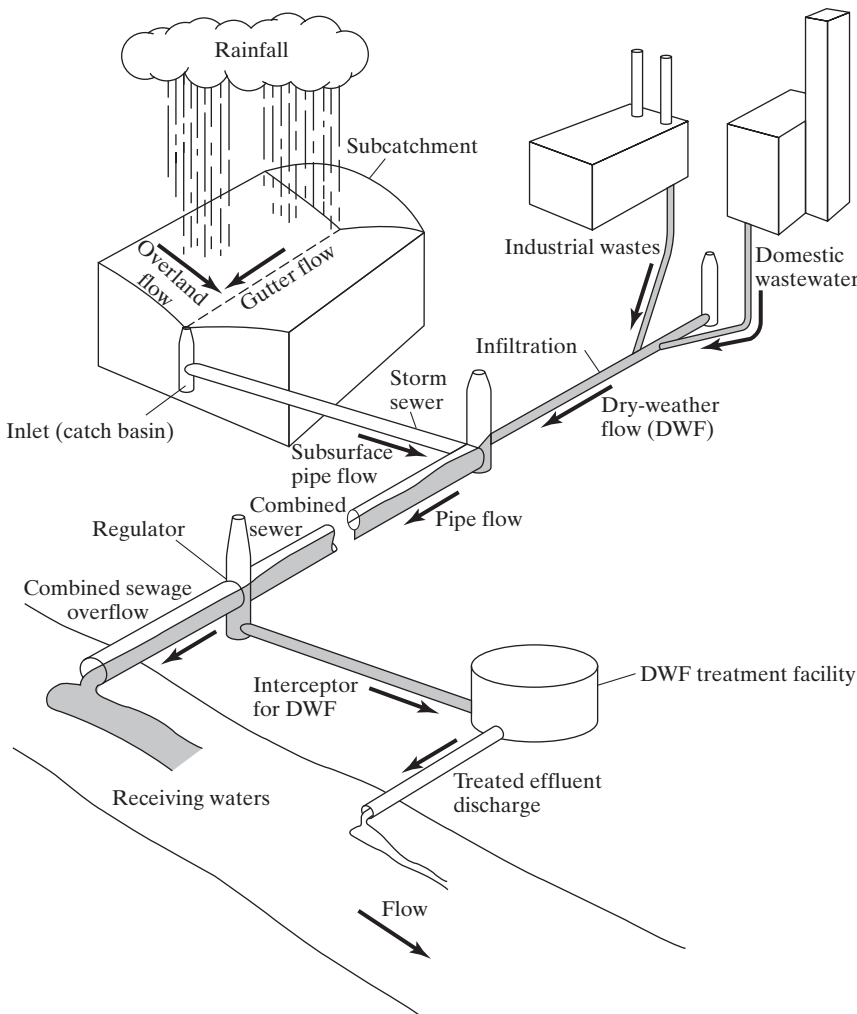


Figure 6–1
Effect of urbanization on urban runoff hydrograph. (a) Shape. (b) Peak flows. (From Leopold, 1968.)

faster response and greater runoff volumes may aggravate the management problem, these same characteristics of urbanization also tend to make the engineering analysis problem somewhat easier, because estimation of losses is simplified and channel characteristics such as shape, slope, and roughness are much better known.

Drainage systems in urban areas may rely on natural channels, but most cities have a sewer network for removal of stormwater. If the system is exclusively for stormwater removal, it is called a **storm sewer**. If the same conduit also carries domestic sewage, it is called a **combined sewer**. A combined sewer usually has a regulator (hydraulic control structure such as a weir or orifice) at its downstream end that diverts the dry-weather flow (domestic and industrial sewage) into an **interceptor**, which carries it to a treatment plant (Fig. 6-2). During wet weather, when the hydraulic capacities of the

Figure 6-2
Urban drainage system. (From Metcalf and Eddy et al., 1971.)



interceptor and the regulator are exceeded, the combined sewer overflow (CSO, a mixture of stormwater and sewage) is released directly to the receiving water body, with the potential for pollution problems. When the stormwater and domestic sewage are carried in separate conduits, the drainage system is said to be **separated**, as opposed to combined. Most newer cities have separate systems, but many older cities (e.g., Boston, Philadelphia, Atlanta, Chicago, San Francisco) retain their combined systems, especially in the northeastern and midwestern United States.

Storm and combined sewers are installed to remove stormwater from the land surface, thus preventing flooding and permitting normal transportation on highways and the like. As such, they are usually designed to handle a peak flow corresponding to a given return period (Chapter 3) according to local regulations (2–10 yr for suburban drainage and 10–50 yr for major highways are typical). It should always be remembered that a storm (or combined) sewer system is a **minor drainage system** for stormwater. That is, if the capacity is exceeded, the runoff will then find an alternative pathway along the surface, or **major drainage system**. If the surface system (streets, for instance) is not designed to accommodate it, then the surface flow may flow through structures, with disastrous consequences. In other words, the major drainage system should always be designed with consideration of the possibility of failure or exceedance of the minor system (see Chapter 9).

The Engineering Problem in Urban Hydrology

The engineering problem in urban hydrology usually consists of the need to control peak flows and maximum depths throughout the drainage system. If the hydraulic grade line is too high, sewers may **surcharge**; that is, the water level may rise above the crown (top) of the sewer conduit, leading occasionally to basement flooding or discharge to streets. New facilities must be designed to minimize such occurrences, and existing drainage systems must often be modified (“**retrofitted**”) to correct for them. Exceeding the capacity of an existing system is a problem that often occurs in newly developed areas that are served by an old sewer system.

The water quality of urban runoff may also be poor (Environmental Protection Agency, 1983; Huber, 1993; Novotny, 1995), and special measures may be required simply to improve the quality of the runoff prior to discharge into receiving waters. This is especially true for the discharge of combined sewer overflows (Water Pollution Control Federation, 1989; Water Environment Federation and American Society of Civil Engineers, 1998).

Alternatives for control of urban runoff quantity are many; a representative list is given in Table 6–1 and discussed later. In new developments, it may be possible to provide retention (e.g., by infiltration) and detention to reduce downstream effects, but this is seldom an economical alternative for remediation of older systems in dense urban settings. Attenuation of flood

Table 6–1 Some Alternatives for Control of Urban Runoff Quantity

Measure	Description
<i>Storage</i>	
Detention	Water detained and released after attenuation by reservoir effect
Retention	Water retained in facility, not released downstream; removed only by infiltration and evaporation
Examples	Basins, concrete tanks, available storage within sewer system, swales, parks, roofs, parking lots
Advantages	Inexpensive if land is available; can promote water conservation and flow equalization
Disadvantages	Costly if land unavailable; storage on private property (e.g., roofs, parking lots) subject to maintenance problems
<i>Increased Infiltration</i>	Reduction of impervious area
Examples	Porous pavement and parking lots
Advantages	Promotes water conservation
Disadvantages	Unsuitable for old cities; unable to reduce volumes of large storms; occasional structural problems
<i>Traditional Flood Control Measures</i>	Measures used in urban and nonurban settings
Examples	Channelization, floodplain zoning, floodproofing
Advantages	Well studied; effective in new areas
Disadvantages	Zoning, floodproofing may be unsuitable for old cities
<i>Other</i>	
Examples	Inlet restrictions, improved maintenance
Comment	Options for older systems

peaks using storage, whether on the surface or within the drainage system, is the most common control alternative. Storage also has the secondary benefit of encouraging infiltration (if the storage basin is on pervious soil) and evaporation, thus reducing the runoff volume as well as the peak. In addition, storage facilities act as sedimentation basins for water quality control. Many additional options are available for quality control (Schueler, 1987; Water Pollution Control Federation, 1989; Schueler et al., 1992; Environmental Protection Agency, 1993; Urbonas and Roesner, 1993; Urbonas and Stahre, 1993; Field et al., 1994; Horner et al., 1994; Water Environment Federation and American Society of Civil Engineers, 1998; Field et al., 2000; Debo and Reese, 2003; Field and Sullivan, 2003; USEPA, 2004; Minton, 2005; Field et al., 2006; Oregon State University et al., 2006).

The American Society of Civil Engineers and other groups maintain a database on the effectiveness of **best management practices** (BMPs). It is available through the World Wide Web at <http://www.bmpdatabase.org/>. **Low impact development**, or LID, is a contemporary site design technique that attempts to mimic predevelopment site hydrology (Wright et al., 2000; Prince George's County, 2000). This emphasis at the parcel (urban lot) level

often means that hydrologic processes must be evaluated on a very small scale. An important concept of LID is emphasis on evapotranspiration for stormwater removal, from such as vegetated (“green” or “eco”) roofs and rain gardens (vegetated swales or depressions receiving runoff from impervious areas) (Oregon State University et al., 2006).

Control options may often be actively traded off against each other during the design phase; when a mixture of controls is used, the cost for one will usually decrease as the use of the other is increased (Heaney and Nix, 1977). Economic optimization is a necessary part of any design. Life-cycle cost analysis of urban drainage systems is discussed by Geosyntec Consultants et al. (2006).

Data and information needed for analysis of problems in urban hydrology are similar to those needed in other areas of hydrology—for example, information on rainfall, surface catchment characteristics, and characteristics of the drainage system, natural or man made. When analytical methods are used to calculate hydrographs, it is highly desirable to have measurements available with which to calibrate and verify predictions (Chapters 2 and 9).

Modern analytical tools range from simple methods for predicting peak flows and runoff volumes to sophisticated computer models for predicting the complete hydrograph at any point in the drainage system. Such methods usually include a means for converting rainfall to runoff from the land surface, followed by a method for routing flows through the drainage system. All standard computer models are available for microcomputers, and most techniques can easily be programmed in a spreadsheet or in a formal language (e.g., FORTRAN, C, BASIC, Java).

Design Objectives

The engineering objective when dealing with urban hydrology is to provide for control of peak flows and maximum depths at all locations within the drainage system while simultaneously protecting the environment. In essence it is the same flood control problem dealt with in Chapter 4, but with the added considerations associated with urbanization. Secondary issues include minimization of increased runoff volumes as well as basement flooding, surcharging, and water quality control.

The analytical problems that must be solved to address these objectives are the predictions of runoff peaks, volumes, and complete hydrographs anywhere in the drainage system. These problems are often separated into those involving the surface drainage system, for which rainfall must be converted into an overland flow hydrograph (or inlet hydrograph, for flow into the inlet to the sewer system), and those involving the channel or sewer system, which often may be handled through conventional flow routing techniques (Chapter 4). However, there is a tendency in urban hydrology to lump the two systems together and produce one aggregate

hydrograph (or peak flow or runoff volume) at the system outlet, such as by a unit hydrograph.

The distinction between the needs to predict peak flows, runoff volumes, and complete hydrographs is important, since the three objectives may require very different analytical methods. In general, predictions of peaks and volumes lend themselves to simplified techniques, while hydrographs require a more comprehensive analysis.

Estimation of base flow in urban drainage systems also requires special consideration, since water may enter the channels both as infiltration (seepage into a conduit or open channel from ground water) and as domestic sewage. The total of the two is often evaluated from monitoring at downstream treatment plants or from special infiltration/inflow (I/I) studies (Environmental Protection Agency, 1977; Pitt et al., 1993).

A further distinction will be made regarding the application of analytical techniques—namely, the design of new systems vs. the alleviation of drainage problems in old ones. More options are usually available for the former, whereas existing systems may impose severe constraints on control options for the latter.

Rainfall–Runoff

Conversion of rainfall into runoff in urban areas is usually somewhat simplified because of the relatively high imperviousness of such areas, although in residential and open-land districts the calculation of infiltration into pervious surfaces may still represent a critical factor in the analysis. Infiltration estimates are complicated by the fact that urban soils are inevitably disturbed, and estimates based on data for natural soils may be in error (Pitt et al., 1999, 2001). Once computed, however, rainfall excess may be converted into a runoff hydrograph using almost any of the methods used for natural catchments (Chapter 2).

When hydrographs are to be computed, special effort is required to obtain adequate rainfall data. This is because urban areas respond quickly to rainfall transients, in contrast to natural catchments, which dampen out short-term fluctuations. Thus, rainfall data should be available at 5-min or shorter increments to predict the runoff hydrograph adequately. Since 5-min data are seldom available, 15-min data from the National Climatic Data Center (NCDC) of the U.S. National Oceanographic and Atmospheric Administration (NOAA) are often the next best resource (Appendix E). A further consideration is the storm direction; as for natural catchments, the hydrograph can have a considerably higher peak if the storm moves down the catchment toward the outlet. This means that one rain gage is seldom sufficient; at least three, plus a wind measurement, may be necessary to describe the dynamics of moving storms (James and Scheckenberger, 1984). Radar provides an excellent means of tracking moving storms (Chapter 11).

6.2 REVIEW OF PHYSICAL PROCESSES

Catchment Description

The urban catchment is characterized by its area, shape, slope, soils, land use, imperviousness, roughness, and storage. The area and imperviousness are the two most important parameters for a good prediction of hydrograph volumes. Although it is a seemingly straightforward parameter, estimation of the percent imperviousness can be subtle. In particular, it is usually necessary to distinguish between **directly connected impervious areas** (DCIA) and those that are not. Directly connected, or hydraulically effective, impervious areas are those that drain directly into the drainage system, such as a street surface with curbs and gutters that directs the runoff into a storm sewer inlet. **Nondirectly connected impervious areas** include rooftops or driveways that drain onto pervious areas. Runoff from such areas does not enter the storm drainage system unless the pervious areas become saturated.

Estimates of imperviousness can be made by measuring such areas on aerial photographs or by considering land use; typical values are shown in Table 6–2. More recent data compiled using a geographical information system (GIS) are presented by Field et al. (2000) and Lee and Heaney (2003). For large urban areas, imperviousness can be estimated on the basis of population density (e.g., Stankowski, 1974):

$$I = 9.6 PD^{(0.573 - 0.017 \ln PD)}, \quad (6-1)$$

where

I = percent imperviousness,
 PD = population density (persons/ac).

Equation (6–1) is based on a regression analysis of 567 communities in New Jersey, so it should be used with caution elsewhere! Imperviousness is often used as a calibration parameter in models.

Superimposed on the catchment is the drainage system, either natural or (more likely) **improved**—that is to say, *hydraulically* improved— aesthetics may be another matter! Improved drainage usually consists of

Table 6–2 Imperviousness by Type of Land Use for Nine Ontario Cities

Land Use	Percent Imperviousness	
	Average	Range
Residential	30	22–44
Commercial	81	52–90
Industrial	40	11–57
Institutional	30	17–38
Open	5	1–14

(Evaluation of the Magnitude and Significance of Pollution from Urban Stormwater Runoff in Ontario, Research Report No. 81, Canada-Ontario Agreement, Environment Canada, 1978. Reproduced with the permission of the Minister of Public Works and Government Services Canada, 2012.)

a network of channels and conduits forming a sewer system that normally flows with a free-surface or open-channel flow, as opposed to pressure flow in a pipeline. The drainage system thus has its own set of parameters to describe its geometry and hydraulic characteristics. For a new system, all of these parameters are design parameters and may be varied according to the engineer's needs. For instance, the design of new systems often emphasizes retention of natural stream functions as much as possible, for ecological and aesthetic reasons. For an existing system, a major effort at collecting data may be required to describe the as-built system configuration accurately. This is especially difficult in old systems in which ad hoc repairs and alterations may have been made over many decades and for which a field survey may be required if accurate data, such as elevations of conduit **inverts** (bottoms), are to be obtained. Accurate surveying is usually a necessary preliminary activity prior to analysis or modeling.

Calculation of Losses

Rainfall excess is computed as rainfall minus losses. Losses result from depression storage (or "initial abstraction") on vegetation and other surfaces, infiltration into pervious surfaces, and evaporation (Chapters 1 and 2). For an individual storm, evaporation is relatively unimportant, but for a long-term analysis of the urban water budget (or for long-term simulation), evaporation is just as important as it is in natural catchments. Infiltration calculations can also proceed in the same manner as for natural areas (Chapter 2), with allowance for the impact of compacted soils on infiltration rates (Pitt et al., 1999, 2001).

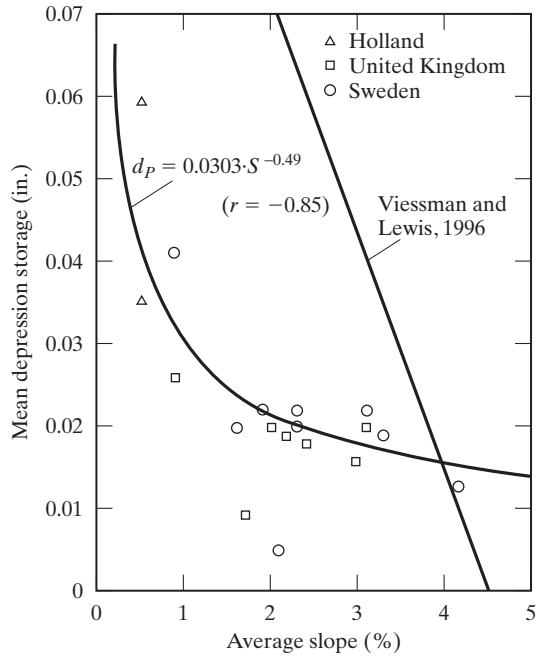
Depression storage is difficult to separate from infiltration over pervious areas; rough estimates for large urban areas include those shown in Table 6–3. For highly impervious areas, measurements have been made to relate depression storage to the slope of the catchment (Fig. 6–3). Depression storage is often used as a calibration parameter in models, since, although representative of a real loss, its estimation is very difficult a priori.

Table 6–3 Depression Storage Estimates in Urban Areas

City	Depression Storage		Reference
	Cover	(in.)	
Chicago	Pervious	0.25	Tholin and Keifer (1960)
	Impervious	0.0625	
Los Angeles	Sand	0.20	Hicks (1944)
	Loam	0.15	
	Clay	0.10	

Figure 6-3

Depression storage vs. catchment slope. (After Kidd, 1978; Viessman and Lewis, 1996.)

**EXAMPLE 6-1****COMPUTATION OF RAINFALL EXCESS**

The rainfall hyetograph shown in Figure E6-1 and listed in Table E6-1 is subject to a depression storage loss of 0.15 in. and Horton infiltration [Eq. (2-42)] with parameters $f_0 = 0.45$ in./hr, $f_c = 0.05$ in./hr, and $k = 1 \text{ hr}^{-1}$. Calculate the hyetograph of rainfall excess.

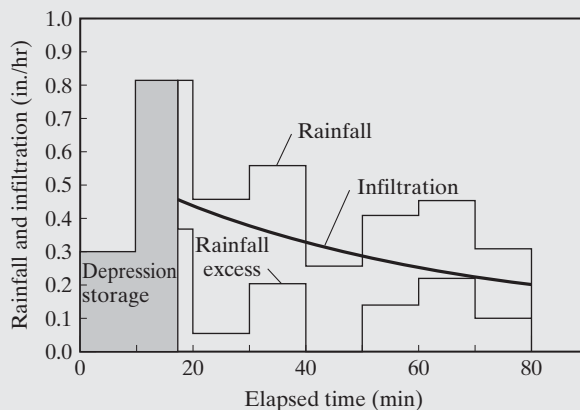
Figure E6-1
Rainfall and rainfall excess hyetographs.

Table E6-1 Computation of Rainfall Excess

Time Interval (min)	Rainfall (in./hr)	Infiltration Capacity at Start of Interval (in./hr)	Average Infiltration Capacity (in./hr)	Average Rainfall Excess (in./hr)
0-10	0.30			
10-17.5	0.80			
17.5-20	0.80	0.450	0.442	0.358
20-30	0.45	0.434	0.404	0.046
30-40	0.55	0.375	0.350	0.200
40-50	0.25	0.325	0.304	0*
50-60	0.40	0.283	0.265	0.135
60-70	0.45	0.247	0.232	0.218
70-80	0.30	0.217	0.204	0.096
80-90	0.00	0.191		0.000

*Negative value set to zero.

Depression storage is customarily removed “off the top,” prior to the deduction of any infiltration losses. The volume of the first 10-min rainfall increment is $0.3 \text{ in./hr} \times 1/6 \text{ hr} = 0.05 \text{ in.}$ This leaves $0.15 - 0.05 = 0.10 \text{ in.}$ of depression storage to be filled during the next rainfall increment(s). This occurs at a time

$$t = 1/6 \text{ hr} + (0.10 \text{ in.}/0.8 \text{ in./hr}) = 1/6 \text{ hr} + 1/8 \text{ hr} = 17.5 \text{ min.}$$

Thus infiltration begins at 17.5 min, as sketched in Figure E6-1. The infiltration calculations are summarized in Table E6-1, and the rainfall excess is sketched in Figure E6-1. Note that a negative rainfall excess is not possible. Note also that for this example, infiltration capacity is assumed to continue to diminish at the same rate between times 40 and 50 min even though the rainfall rate is less than the infiltration capacity. The integrated form of Horton’s equation may be used to avoid this situation, as discussed in Chapter 2.

The volumes of rainfall and rainfall excess are found by summing the ordinates and multiplying by the time interval ($1/6 \text{ hr}$). Thus, the rainfall volume is $3.5 \times 1/6 = 0.583 \text{ in.}$ The volume of rainfall excess is computed similarly, except that the first time increment is only 2.5 min instead of 10 min. Thus, the volume of rainfall excess is $0.695 \times 1/6 + 0.358 \times 2.5/60 = 0.131 \text{ in.}$ Since the depression storage volume was 0.15 in., the actual infiltration volume is $0.583 - 0.15 - 0.131 = 0.302 \text{ in.}$ Note that this is *not* equal to the area under the infiltration curve, since not all of the infiltration capacity was used during the interval between 40 and 50 min. A volumetric runoff coefficient can be defined simply as the ratio of runoff (rainfall excess) to rainfall. Here the ratio is $0.131/0.583 = 0.22$.

When rainfall excess is computed, it is customary to leave it tabulated in the original hyetograph time increments (10 min for this example). In

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this example, there is a complication for the first interval, since it begins at 17.5 min. There are at least two options: (1) the starting time could remain at 17.5 min, and if equal time intervals are needed, an interval of 2.5 min could be used; (2) the first short interval of rainfall excess could be averaged over the time interval from 10 to 20 min. This would yield a rainfall excess value of $131/0.583 = 0.22$ for the interval from 10 to 20 min (and zero for the first interval).

Time of Concentration

The time of concentration t_c was introduced in Chapter 4 as part of the development of kinematic wave theory. The definitions of t_c are worth a brief review, since it is fundamental to much of the analysis in urban hydrology. There are two related definitions:

1. The time of concentration is the travel time of a *wave* to move from the hydraulically most distant point in the catchment to the outlet.
2. The time of concentration is the time to equilibrium of the catchment under a steady rainfall excess (i.e., when the outflow from the catchment equals the rainfall excess onto the catchment).

Note especially that t_c is *not* the travel time taken by a parcel of water to move down the catchment, as is so often cited in texts. The catchment is in equilibrium when the time t_c is reached because the outlet then “feels” the inflow from every portion of the catchment. Since a wave moves faster than does a parcel of water, the time of concentration (and equilibrium) occurs sooner than if based on overland flow (or channel) water velocities (Overton and Meadows, 1976). For overland flow, the wave speed (celerity) is usually given by the kinematic wave equation

$$c = mV = \alpha y^{m-1}, \quad (6-2)$$

where

- c = wave speed,
- V = average velocity of water,
- y = water depth,
- α, m = kinematic wave parameters in the uniform flow momentum equation, and

$$q = yV = \alpha y^m, \quad (6-3)$$

where

- q = flow per unit width of the catchment.

Using Manning’s equation, the parameter m is $5/3$ for turbulent flow and 3 for laminar flow. Thus, the wave can travel from 1.7 to 3 times as fast as does

a parcel of water. Corresponding values of the parameter α for turbulent and laminar flow, respectively, are

$$\alpha = \frac{k_m \sqrt{S}}{n} \quad (m = 5/3) \quad \text{turbulent flow,} \quad (6-4)$$

$$\alpha = gS/3\nu \quad (m = 3) \quad \text{laminar flow,} \quad (6-5)$$

where

- S = slope (small, such that $\sin S \approx \tan S \approx S$),
- n = Manning's roughness,
- g = acceleration of gravity,
- ν = kinematic viscosity, and
- $k_m = 1.49$ for units of ft and s in Equation (6-4),
 $= 1.0$ for units of m and s .

The time of concentration for overland flow by kinematic wave theory is

$$t_c = \left(\frac{L}{\alpha i_e^{m-1}} \right)^{1/m}, \quad (6-6)$$

where

- L = length of overland flow plane,
- i_e = rainfall excess.

Care should be taken to use consistent units in Equation (6-6)! It can be seen that t_c depends inversely on the rainfall excess; thus, higher-intensity rainfall excess will reduce t_c . Alternative equations for t_c based on water parcel travel times seldom include this inherent physical property of the catchment (Huber, 1987; McCuen et al., 1984).

KINEMATIC WAVE FLOW COMPUTATION

EXAMPLE 6-2

Water flows down an asphalt parking lot that has a 1% slope. What are the values of α and m [Eq. (6-3)] for the following two cases:

- (a) laminar flow ($T = 20^\circ\text{C}$),
- (b) turbulent flow ($n = 0.013$ from Table 4-2).

What is the flow per unit width q (cfs/ft) for each of the two cases for a water depth of 0.3 in.?

Laminar flow has an exponent $m = 3$ and a coefficient α , given by Equation (6-5). At a temperature of 20°C , $\nu = 1.003 \times 10^{-2} \text{ cm}^2/\text{s} = 1.080 \times 10^{-5} \text{ ft}^2/\text{s}$ (Table C-1 in Appendix C). Thus,

SOLUTION

$$\alpha = gS/3\nu = \frac{32.2 \times 0.01}{3 \times 1.080 \times 10^{-5}} = 9938 \text{ ft}^{-1}\text{s}^{-1}.$$

For turbulent flow, the exponent $m = 5/3$, and from Equation (6-4),

$$\alpha = \frac{1.49S^{0.5}}{n} = \frac{1.49 \times 0.01^{0.5}}{0.013} = 11.46 \text{ ft}^{1/3}/\text{s}.$$

A depth of 0.3 in. = 0.025 ft. The flow per unit width is given by Equation (6-3):

$$\text{Laminar: } q = 9938 \times 0.025^3 = 0.155 \text{ cfs/ft},$$

$$\text{Turbulent: } q = 11.46 \times 0.025^{5/3} = 0.024 \text{ cfs/ft}.$$

For the same depth, the laminar flow rate is higher, but in most actual cases turbulent flow is observed at the downstream end of the overland flow plane (Emmett, 1978).

In free-surface flow, waves in channels and conduits (e.g., street gutters, sewers) move with the *dynamic wave speed*

$$c = V \pm \sqrt{gA/B}, \quad (6-7)$$

where

A = cross-sectional area in the channel,

B = surface width.

Downstream wave speeds (positive sign) given by Equation (6-7) are obviously greater than the water velocity V . Wave speeds calculated using the negative sign are downstream for supercritical flow and upstream for subcritical flow. Thus, dynamic waves can account for backwater effects, whereas kinematic waves move only downstream. If flow through closed conduits (pressure flow) occurs in the catchment, waves move with the speed of sound (Wylie and Streeter, 1978)—very fast indeed.

To summarize, any catchment, urban or otherwise, responds to an input of rainfall through the passage of *waves* downstream along all water pathways—overland flow, open channels and conduits, and closed conduits. The net effect is that the time of concentration (or time to equilibrium) is reached well in advance of the travel time required by a parcel of water to move the complete distance downstream. An accurate value of t_c is essential in the use of the rational method, presented in Section 6.4, for estimation of peak flows as well as for evaluation of the catchment response to changes in the rainfall regime.

Lag Times

Other timing parameters of the urban hydrograph include the various possible lag times discussed in Chapter 2 (Fig. E2-7). These lag parameters, such as time to peak t_p , have the distinct advantage of being possible to measure,

whereas the time of concentration is very difficult to measure (Overton and Meadows, 1976) and is more of a conceptual parameter. Lag times are often incorporated into UH theory and conceptual models.

Flow Routing

Flow routing through the urban drainage system may be accomplished using almost any of the methods discussed in Chapter 4. The task is made somewhat easier because of the presence of uniform channel sections (e.g., circular pipes) in the sewer system. But at the same time, difficulties may arise at the many junctions and hydraulic structures encountered in a sewer system. Most of these difficulties may be overcome through the use of more sophisticated hydraulic methods discussed in Section 6.5 (such as the Saint Venant equations) if desired (Yen, 1986); the level of the analysis should be tailored to the desired accuracy of the predictions, and all predictions should be verified against measured flows wherever possible.

Data Sources

Rainfall is the driving force in most hydrologic analysis. Generally, for urban hydrology, shorter time increment data (15-min or less) are better because of the shorter response times of urban basins. Two types of rainfall data are commonly required in urban hydrology: (1) “raw” point precipitation data (i.e., actual hyetographs) and (2) processed data, usually in the form of frequency information. Computerized point precipitation data are available from the NCDC (Asheville, North Carolina 28801-2696, <http://www.ncdc.noaa.gov/>) in the form of hourly or 15-min values from recording rain gages. Recently, 5-min data are available for a number of stations, with the period of record beginning at about the year 2000 (<ftp://ftp.ncdc.noaa.gov/pub/data/asos-fivemin/>). The hourly data were historically given in the monthly “Climatological Data,” published for each state by the National Weather Service (NWS). Photocopies of some older rain gage charts are available for desired dates from the NCDC if more detailed time resolution is needed. The NWS network of weather stations is the prime source for all meteorological data, but its data may often be supplemented by other sources that collect data continuously or as part of special projects. These sources include the U.S. Geological Survey (USGS), the Natural Resources Conservation Service (NRCS), and the Army Corps of Engineers (ACOE) as well as state agencies, the state climatologist, local utilities, and universities. Finally, radar rainfall data may be used, as described in Chapter 11. It is often necessary to contact several agencies to obtain all available data for a catchment.

It is useful to note that the rainfall data published in the NWS’s printed publications are often slightly different from the computerized data obtained from the NCDC because the values come from different rain gages. These

6.3 RAINFALL ANALYSIS IN URBAN BASINS

gages may be within a few feet of each other, but still the recording rain gage data (which are computerized) almost always differ from the nonrecording gage data (which are printed as daily totals in official publications). The nonrecording (printed) data are traditionally regarded as “official.”

Processed data include many varieties of statistical summaries, but the most common type of processed data used in urban hydrology is in the form of intensity–duration–frequency (IDF) curves, shown in Figure 1–15 and later in this chapter. IDF curves are available from the NWS (Hershfield, 1961 and updated at: <http://hdsc.nws.noaa.gov/hdsc/pfds/index.html>), the NRCS, and local sources such as state departments of transportation (e.g., Weldon, 1985; Oregon Department of Transportation, 2005). The best information usually comes from local sources, an increasing amount of which may be found on the web, e.g., by a simple search or using the following NOAA site as a starting point: <http://www.nws.noaa.gov/ohd/hdsc/index.html>.

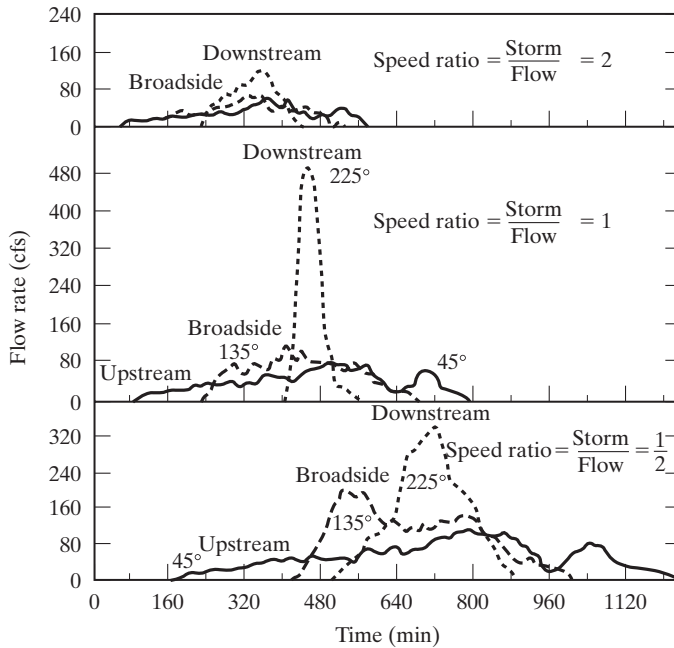
Rainfall Measurement

As mentioned earlier, a unique aspect of urban hydrology is the fast response of an urban catchment to the rainfall input. This means that high-frequency information in the rainfall signal is transformed into high-frequency pulses in the runoff hydrograph, since the highly impervious urban catchment does not significantly dampen such fluctuations. In most cases, a tipping-bucket rain gage will provide an adequate resolution of high frequencies, in contrast to weighing bucket gages, which are also commonly employed. Tipping-bucket gages (Chapter 1) are much easier to install in a network because the electrical pulses can be transmitted over a telephone or other communication device or recorded on a data logger at the site. In most urban modeling work, it is necessary to have hyetographs with 5-min or better resolution, although this may be relaxed somewhat for larger basins.

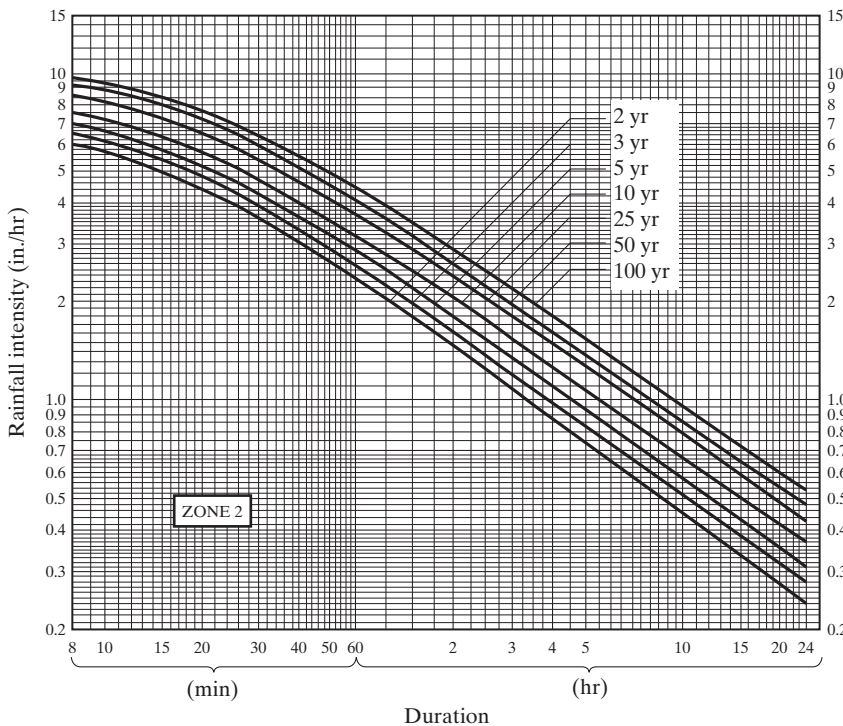
Unless the catchment is very small (such that a storm completely “covers” the basin spatially), the outfall hydrograph will also be sensitive to the storm direction (James and Shtifter, 1981). To simulate the effects of a moving (or kinematic) storm, it is necessary to have more than one rain gage so that the longitudinal movement (up or down the basin) can be defined. Movement of the storm *across* the flow direction does not affect the hydrograph as much as longitudinal movement. These effects can be seen in Figure 6–4.

Intensity–Duration–Frequency Curves: Use and Misuse

IDF curves summarize *conditional* probabilities (frequencies) of rainfall depths or average intensities. Specifically, IDF curves are graphical representations of the probability that a certain average rainfall intensity will occur, given a duration; their derivation is discussed by McPherson (1978). For example, IDF curves for Houston, TX, are shown in Figure 1–15 and for Tallahassee, Florida, are shown in Figure 6–5 (Weldon, 1985). The return

**Figure 6-4**

Effect of storm moving upstream, downstream, and broadside to an urban catchment. (From Surkan, 1974.)

**Figure 6-5**

IDF curves for the Tallahassee, Florida, region. (Author: K. E. Weldon, in "FDOT Rainfall Intensity_Duration_Frequency Curve Generation," in *Stormwater Management, An Update*, M. P. Wanielista and Y. A. Yousef, eds., Environmental Systems Engineering Institute, University of Central Florida, Orlando, FL, July, pp.11-31, © 1985)

period of an average intensity of 3.6 in./hr for a duration of 40 min in Tallahassee is 5 yr. (As described in Chapter 3, the return period is the reciprocal of the probability that an event will be equaled or exceeded in any one year or other time unit.) Conversely, if one wanted to know what average intensity could be expected for a duration of 20 min once, on the average, every 10 yr, Figure 6-5 gives a value of 5.6 in./hr. These values are conditional on the duration—the duration must always be specified by some means to use IDF curves—and the duration is *not* the duration of an actual rainstorm.

A critical characteristic of IDF curves is that the intensities are indeed averages over the specified duration and *do not represent actual time histories of rainfall*. The contour for a given return period could represent the smoothed results of several different storms. Moreover, the duration is not the actual length of a storm; rather, it is merely a 20-min period, say, within a longer storm of any duration, during which the average intensity happened to be the specified value. In fact, given that the IDF curves are really smoothed contours, unless a data point falls on a contour, they are completely hypothetical! A detailed analysis of the method of constructing IDF curves and the perils of improper interpretation is given by McPherson (1978).

To reiterate, (1) IDF curves *do not* represent time histories of real storms—the intensities are *averages* over the indicated duration, (2) a single curve represents data from several different storms, (3) the duration is *not* the duration of an actual storm and most likely represents a shorter period within a longer storm, and (4) it is incorrect to use IDF curves to obtain a storm event volume because the duration must be arbitrarily assigned. The preponderance of IDF information in the hydrologic literature is primarily a result of the need for IDF curves for use in the rational method (described in Section 6.4). As will be seen, the IDF data are properly applied in this case.

Definition of a Storm Event

Return periods (or frequencies) can be assigned to rainfall events on the basis of several different parameters, but most commonly on the basis of total volume, average intensity, peak intensity, duration, or interevent time. To work with any of these parameters, the rainfall time series must first be separated into a series of discrete, independent events. When this is done, they may be ranked by volume or any desired parameter and a conventional frequency analysis performed (Chapter 3).

For ease of computation, a statistical measure is usually employed to separate independent storm events. A minimum interevent time (MIT) is defined such that rainfall pulses separated by a time less than this value are considered part of the same event. The process can be illustrated by an example.

SEPARATION OF RAINFALL INTO EVENTS**EXAMPLE 6-3**

Given the hypothetical rainfall sequence shown in Figure E6-3, determine the number of events corresponding to various values of the MIT.

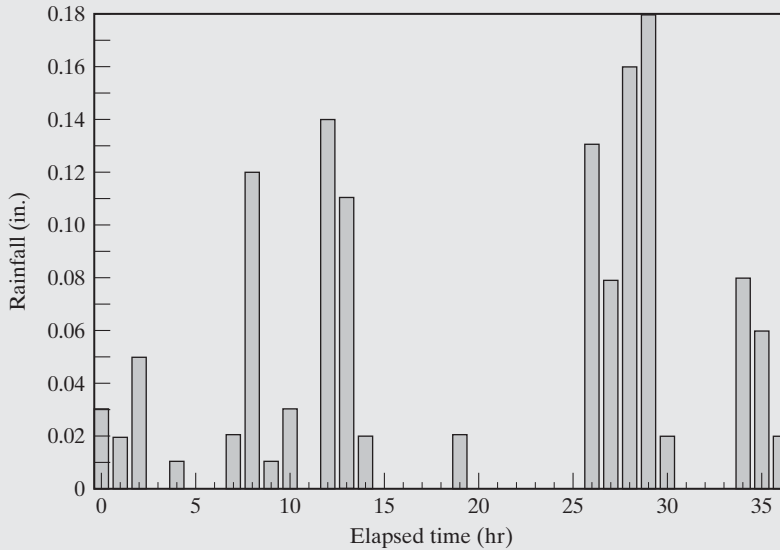


Figure E6-3
Hypothetical
hourly rainfall
hyetograph for
illustration of
event separation.

The results are summarized in the following table:

SOLUTION

MIT	Number of Events
0	20
1	7
2	5
3	4
4	3
5	2
6	2
≥ 7	1

When an MIT value of zero is used, all rainfall hours are considered to be separate events; hence the number of events is equal to the number of rainfall hours. For an MIT value of 1 hr, at least 1 dry hour is required to separate events, leading to the seven events (clusters of contiguous wet hours) given in the table. Since there are no rainfall hours separated by more than 6 hr, all rainfall is considered to be a single event for an MIT value greater than or equal to 7 hr. Note that the magnitudes of the hourly rainfall values play no role in this analysis unless a threshold depth is used, such as 0.10 in., below which it is assumed no runoff will occur.

Several possibilities exist for a statistical measure with which to define independence between rainfall values, including examination of the correlation between rainfall values at different lags (Heaney et al., 1977; Tavares, 1975). However, the easiest definition (Hydroscience, 1979; Restrepo-Posada and Eagleson, 1982) is based on the observation that interevent times are usually well described by an exponential probability density function (Section 3.5), for which the standard deviation equals the mean (or their ratio, the coefficient of variation CV , equals 1.0). The procedure is to use trial values of MIT until the CV of the interevent times is closest to 1.0. Typical values of the resulting MIT may range from 5 to 50 hr for a time series of hourly rainfall values, depending on location and season. The analysis is facilitated by the use of a computer program such as SYNOP (Driscoll et al., 1989; Environmental Protection Agency, 1976; Hydroscience, 1979), also integrated into the Statistics Tool of the SWMM5 model (<http://www.epa.gov/nrmrl/wswrd/wq/models/swmm/>), which performs the event separation and frequency analysis for a given MIT value, in the manner of Example 6–3.

Having completed the event separation, the events may be ranked by the parameter of interest for any desired frequency analysis. For example, the seven highest total storm volumes for a 25-yr record of hourly rainfalls in Tallahassee, Florida, are shown in Table 6–4, for which the MIT is 5 hr. It is clear that these volumes can arise from actual storms of greatly differing durations, certainly not fixed as on an IDF curve. For comparison, 24-hr depths are also shown, taken from IDF data for the Tallahassee region (Weldon, 1985). Notice that the 24-hr depths are in the correct range of return periods for total storm depths, but this would not be true if other durations were used.

Table 6–4 Comparison of 24-hr IDF Storm Depths with SYNOP Frequency Analysis of Historical Storms for Tallahassee, Florida, 1949–1975

Storm Date	Return Period* (yr)	Duration (hr)	Depth (in.)	Intensity (in./hr)
9/20/69	26.0	54	13.41	0.25
IDF	25	24	10.08	0.42
7/17/64	13.0	33	9.76	0.30
IDF	10	24	8.64	0.36
12/13/64	8.7	36	9.73	0.27
7/28/75	6.5	53	8.84	0.17
7/21/70	5.2	20	8.18	0.41
IDF	5	24	7.32	0.31
8/30/50	4.3	36	7.34	0.20
6/18/72	3.7	46	7.17	0.16

*By Weibul formula [Eq. (3–75)], actual return period will be different if based on a fitted distribution (Chapter 3).

Choice of Design Rainfall

Often in an urban analysis it is necessary to provide a design storm for a model with which to evaluate the effectiveness of a drainage system. Ideally these rainfall inputs consist of measured hyetographs obtained during a monitoring program of the catchment and are used for calibration and verification of a model. In this case, there is little question about the choice of rainfall input, but more often than not an agency will require a design storm corresponding to a specified return period, such as a 25-yr storm. There are several ways to define such an event, and the method of constructing and applying a design storm can be very controversial (Adams and Howard, 1985; Harremšeos, 1983; McPherson, 1978; Patry and McPherson, 1979; Debo and Reese, 2003).

The first question is, to what storm parameter does the return period apply? That is, is it a 25-yr storm based on runoff volume, peak flow, average flow, or rainfall volume, rainfall average intensity, and so on? Equally important, given that antecedent conditions in the catchment can be highly variable and that the catchment will alter the nature of the rainfall input in any case, the return period of a storm based on rainfall characteristics will *not*, in general, be the same as the return period of the same storm based on runoff characteristics. For example, the 25-yr storm based on rainfall volume may be only the 5-yr storm based on runoff volume if the catchment is dry before the storm occurs. (This characteristic is illustrated in the SWMM5 example at the end of this chapter.) Thus, it is an error to assign a return period to a runoff characteristic based on a frequency analysis of rainfall.

Synthetic Design Storms

The use of IDF curves (e.g., Fig. 6–5) is intimately linked to most of the methods for constructing **synthetic** (hypothetical) **design storms**. A typical synthetic design storm is constructed in the following manner (e.g., King County, 1995; Debo and Reese, 2003). First, a duration is specified, often arbitrarily by an agency, say 24 hr. Second, the 24-hr depth is obtained from an IDF curve for the desired return period. Third, the rainfall must be distributed in time to construct the synthetic hyetograph. There are major differences in the literature in this last step (Arnell, 1982). The shape varies, mainly depending on the type of storm to be simulated: A cyclonic storm usually has the highest intensities near the middle, and a convective storm usually has the highest intensities near the beginning. However, in the United States, temporal distributions published by the Natural Resources Conservation Service (formerly the SCS) are often used, such as the type II distribution in the Southeast and type 1A distribution in the Pacific Northwest.

EXAMPLE 6-4**CONSTRUCTION OF SCS TYPE II, 24-HR DESIGN STORM**

Construct a 5-yr, 24-hr design storm for Tallahassee, using the SCS type II distribution.

SOLUTION

The distribution (percentage mass curve) is tabulated in Table E6-4. The duration of the design storm must be 24 hr to use the SCS type II distribution, which is specifically given for a 24-hr duration. From the Tallahassee IDF curve shown in Figure 6-5, the average intensity for 24 hr and a 5-yr return period is 0.305 in./hr. This leads to a total depth for 24 hr of 7.32 in., as indicated in Table 6-4. The total depth is allocated over the hourly increments in Table E6-4, and the average intensities for each hour are plotted in Figure E6-4. Because the time interval used is 1 hr, average intensities are the same numerically as incremental depths, which are not shown. Notice the very high intensity during hour 12, reflecting the fact that 42.5% (i.e., 66% - 23.5%, from Table E6-4) of the total rainfall is assumed to occur during this interval.

Table E6-4 24-hr Design Storm from SCS Type II Distribution (Total Depth = 7.32 in.)

Hour	Total Depth (%)	Cumulative Depth (in.)	Average Intensity (in./hr)
0	0.00	0.00	
1	1.00	0.07	0.07
2	2.20	0.16	0.09
3	3.55	0.26	0.10
4	4.91	0.36	0.10
5	6.20	0.45	0.09
6	8.10	0.59	0.14
7	10.00	0.73	0.14
8	12.10	0.89	0.15
9	14.70	1.08	0.19
10	18.60	1.36	0.29
11	23.50	1.72	0.36
12	66.00	4.83	3.11
13	77.40	5.67	0.83
14	82.10	6.01	0.34
15	85.30	6.24	0.23
16	88.10	6.45	0.20
17	90.10	6.60	0.15
18	92.20	6.75	0.15
19	93.50	6.84	0.10
20	94.80	6.94	0.10
21	96.10	7.03	0.10
22	97.40	7.13	0.10
23	98.70	7.22	0.10
24	100.00	7.32	0.10

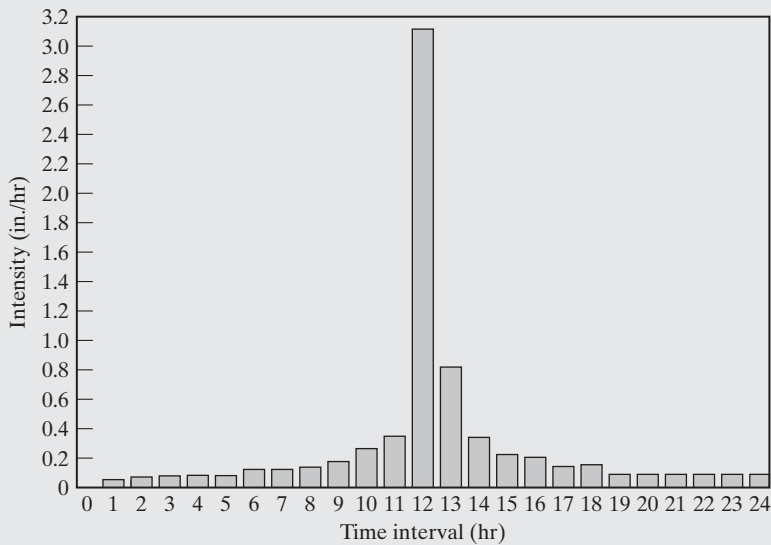


Figure E6-4
SCS type II, 5-yr,
24-hr design
storm for
Tallahassee,
Florida.

McPherson (1978) compares a 5-yr, 3-hr synthetic storm for Chicago with historic storms based on total storm event depth. As shown in Figure 6-6, not only is the depth of the synthetic storm different from the two historic storms that bracket a 5-yr return period, but also the hyetograph shapes bear no relation to the assumed shape for the synthetic storm. Thus, although the synthetic storm may certainly be applied in a study, the shape of the storm is so unusual and its duration so arbitrary that the true return period of the storm, based on any criterion, is totally unknown.

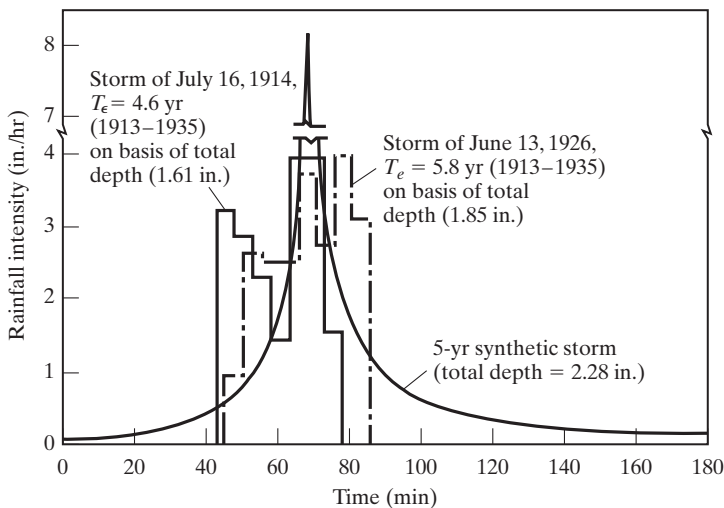


Figure 6-6
Actual vs. synthetic
storm patterns,
Chicago. (From
McPherson, 1978.)

Alternatives to Synthetic Design Storms

Synthetic design storms are very popular because they are relatively easy to construct and use. They require only IDF curves and an assumed shape for the hyetograph, not extensive historic rainfall data, and they may be consistently applied by oversight agencies. What are the alternatives? The best method is to analyze historical runoff data for the catchment to identify storms of interest based on a frequency analysis of peak flows, flood depths, runoff volumes, and so on. The storms that caused, say, the 5-yr peak flow could then be used for design purposes (given adequate information on antecedent conditions). This would involve several storms, with very different temporal distributions, each with a return period close to 5 yr, based on some criterion. The case study at the conclusion of this chapter illustrates a collection of such storms, based on simulation results, not monitoring results.

Selection of storms from a monitoring program is seldom possible in urban areas because flow-monitoring programs are typically short, and there may be extensive development underway in the catchment that alters the rainfall–runoff mechanisms, thus destroying the homogeneity of the runoff record. Probably the best alternative is to apply a continuous simulation model to simulate runoff hydrographs for a period of, say, 25 yr based on historic rainfall records available from the NCDC. When available, 15-min data are preferable to the more common 1-hr data. The model must first be calibrated using measured rainfall and runoff data. A frequency analysis can then be performed on the simulated hydrographs—on the parameters of interest—from which historical design storms can be identified for more detailed analysis (Huber et al., 1986). (The detailed analysis must also have a means for establishing the correct antecedent conditions, a problem automatically handled during the continuous simulation.) Both of these options (frequency analysis of measured or simulated hydrographs) have the advantage of an analysis of runoff, not of rainfall. This continuous simulation approach is routinely used by the USGS to develop its regional flood frequency data and is demonstrated in the case study at the end of this chapter.

A third alternative is a frequency analysis on rainfall *events*, similar to that shown in Table 6–4. If historical events from such an analysis are subsequently used for design storms, the disadvantage remains of basing return periods on rainfall characteristics instead of on runoff characteristics, but at least there would not be an assumption of a duration. A frequency analysis of rainfall statistics can also be used in a **derived distribution** approach, in which the frequency distribution of runoff is derived from the frequency distribution of rainfall (Adams and Papa, 2000).

To summarize, the use of historical storms for design purposes is preferred because the frequency analysis can be directed to the specific parameter of interest without an assumed duration and because there is no need to make an arbitrary assumption about the shape of the hyetograph. The use of historical storms also has an advantage when dealing with the public, because a design can be presented to prevent the flooding that occurred

from a specific real event in the public's memory. The main disadvantage is simply the extra effort involved in a continuous simulation or a frequency analysis based on storm events rather than on conditional frequencies readily available from published IDF data. As a result, synthetic design storms are most often used in practice.

A final word of caution is warranted regarding continuous simulation. Because there are seldom enough long-term rain gages to cover the urban area adequately, a single gage is often used to drive a continuous model, assuming that the entire catchment receives the point rainfall measured by the gage. This is an acceptable assumption for small catchments and/or spatially uniform (e.g., cyclonic) rainfall. However, for spatially variable (e.g., convective) rainfall, peaks may be overestimated by the model because the rainfall may not cover the entire basin. Long-term volumes are probably adequately reproduced, but peak flows identified from a frequency analysis using a modeling scheme of this sort should be reexamined on an event basis using a realistic spatial distribution of rainfall before being used for design purposes.

Peak Flow, Volume, or Hydrograph?

Recall that there are several possible parameters to be determined in an urban hydrologic analysis, but most often they include peak flow, runoff volume, or the complete runoff hydrograph. Related parameters might be the hydraulic grade line or flooding depths. If hydrographs are predicted, then peaks and volumes are an implicit part of the analysis, but some of the simpler methods, e.g., the rational method, will not necessarily provide all parameters of possible interest.

6.4 METHODS FOR QUANTITY ANALYSIS

Peak Flows by the Rational Method

The rational method dates from the 1850s in Ireland (Dooge, 1973) and is called the Lloyd-Davies method in Great Britain. Published in the United States by Kuichling (1889), it is one of the simplest and best-known methods routinely applied in urban hydrology; although it contains subtleties that are not always appreciated. Peak flows are predicted by the simple product

$$Q_p = k_c C i A, \quad (6-8)$$

where

- Q_p = peak flow (cfs or m³/s),
- C = runoff coefficient,
- i = rainfall intensity (in./hr or mm/hr),
- A = catchment area (ac or ha),
- k_c = conversion factor.

When U.S. customary units are used, the conversion factor $k_c = 1.008$ to convert ac-in./hr to cfs and is routinely ignored; this simple conversion is the basis for the term *rational* in the rational method. (The approximate equivalence of ac-in./hr to cfs is worth remembering for convenient rough calculations.) For the alternative metric units given with Equation (6-8), the conversion factor $k_c = 0.00278$ to convert ha-mm/hr to m³/s.

Because of the assumptions regarding the homogeneity of rainfall and equilibrium conditions at the time of peak flow, the rational method should not be used on areas larger than several hundred acres (a few hundred hectares) without subdividing the overall catchment into subcatchments and including the effect of routing through drainage channels (Water Environment Federation and American Society of Civil Engineers, 1992). Since actual rainfall is not homogeneous in space and time, the rational method becomes more conservative (i.e., it overpredicts peak flows) as the area becomes larger.

All catchment losses are incorporated into the runoff coefficient C , which is usually given as a function of land use (Table 6-5). The runoff coefficient is higher for higher return period (more severe) storms because infiltration and depression storage are relatively less important; values in Table 6-5 are typical for return periods of 2 to 10 yrs. A study for the Denver Urban Drainage and Flood Control District (Wright-McLaughlin Engineers, 1984) documented the effect qualitatively but not quantitatively. That is, runoff coefficients may be increased beyond the upper bounds in Table 6-5 for $T > 10$ yr, but by what factor is not known. Runoff coefficients for a variety of nonurban lands may be found in Schwab et al. (1971).

When multiple land uses are found within the catchment, it is customary to use an area-weighted runoff coefficient in Equation (6-8). A better C -estimate would be obtained from measurements, but peak flow measurements are seldom available to correlate with rainfall intensity. It is often assumed (but is seldom true) that C is approximately the same as the volumetric runoff coefficient, C_{vol} ,

$$C_{vol} = \frac{\text{runoff volume}}{\text{rainfall volume}}. \quad (6-9)$$

When tabulated for actual storm data, there is usually considerable variation in C_{vol} values, but the calculation is useful since it gives a rough idea of catchment losses. One of the principal fallacies of the rational method is the assumption that a real catchment will experience a constant fractional loss, regardless of the total rainfall volume or antecedent conditions. The approximation is better as the imperviousness of the catchment increases; Equation (6-8) is probably most nearly exact for a rooftop.

The intensity i is obtained from an IDF curve (e.g., Fig. 6-5) for a specified return period under the assumption that the duration t_r equals the time of concentration t_c . This is physically realistic because the time of concentration also is the time to equilibrium, at which time the whole catchment contributes to flow at the outfall. Thus, if $t_r < t_c$, then equilibrium would not

Table 6-5 Typical Runoff Coefficients for 2-yr to 10-yr Frequency Design

Description of Area	Runoff Coefficients
<i>Business</i>	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
<i>Residential</i>	
Single-family areas	0.30–0.50
Multi-units, detached	0.40–0.60
Multi-units, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
<i>Industrial</i>	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
<i>Streets</i>	
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
<i>Lawns, sandy soil</i>	
Flat, 2%	0.05–0.10
Average, 2%–7%	0.10–0.15
Steep, 7%	0.15–0.20
<i>Lawns, heavy soil</i>	
Flat, 2%	0.13–0.17
Average, 2%–7%	0.18–0.22
Steep, 7%	0.25–0.35

These runoff coefficients are typical values for return periods of 2–10 yr.

Higher values are appropriate for higher return periods.

Source: ASCE and WPCF (1969).

be reached and it would be wrong to use the total area A . If $t_r > t_c$, then equilibrium would have been reached earlier and a higher intensity should be used. The key is a proper evaluation of t_c . Since it is inversely related to the rainfall intensity, the kinematic wave equation should be used for overland flow. Since t_c by Equation (6-6) is proportional to the (unknown) rainfall intensity, an iterative solution is necessary, combining Equation (6-6) with the IDF curves. Such an iterative solution is easily accomplished, as will be illustrated in Example 6-6.

IDF curves may sometimes be approximated by the following functional form (Meyer, 1928):

$$i = \frac{a}{b + t_r}, \quad (6-10)$$

where a and b are regression coefficients obtained graphically or by least squares. When substituted into Equation (6-6), the equation can rapidly be solved for $t_r = t_c$ by Newton–Raphson iteration (Chapra and Canale, 2002) or through Excel’s Solver function.

To avoid the complications of an iterative process, constant overland flow inlet times are often used to approximate the time of concentration. These vary from 5 to 30 min, with 5 to 15 min being most commonly used (American Society of Civil Engineers and Water Pollution Control Federation, 1969). A value of 5 min is appropriate for highly developed, impervious urban areas with closely spaced stormwater inlets, increasing from 10 to 15 min for flat developed urban areas of somewhat lesser density. Values of 20 to 30 min are appropriate for residential districts with widely spaced inlets, with longer values for flatter slopes and larger areas. Inlet times are illustrated in Problems 6.19 and 6.20. Due to the ease with which iterative computations can be performed on the computer (Example 6-6), there should be little need for simplifications such as inlet times. A detailed rational method example for drainage design is illustrated in Chapter 9. Parameter estimation for wave travel times is illustrated as part of the SWMM5 case study at the end of this chapter.

Another option for avoiding an iterative solution of the coupled kinematic wave equation (6-6) and IDF curve is to use Equation (6-6) with an assumed, representative rainfall intensity, held constant (e.g., Oregon Department of Transportation, 2005). This provides a more realistic estimate of a constant t_c than does the use of inlet times but is less accurate than the full iterative solution.

EXAMPLE 6-5

RATIONAL METHOD DESIGN WITH INLET TIMES

The 10-yr peak flow at a stormwater inlet in Tallahassee, Florida, is to be determined for a 40-ha area in rolling terrain. An inlet time of 20 min may be assumed. Land use is as follows.

Land Use	Area (ha)	Runoff Coefficient*
Single-family residential	30	0.40
Commercial	3	0.60
Park	7	0.15

*From Table 6-5.

We find an area-weighted runoff coefficient:

SOLUTION

$$\bar{C} = \frac{\sum A_i C_i}{\sum A_i} = \frac{30 \cdot 0.4 + 3 \cdot 0.6 + 7 \cdot 0.15}{30 + 3 + 7} = 0.37.$$

From Figure 6-5, for an inlet time of 20 min and return period of 10 yr, the average intensity is 5.6 in./hr (142 mm/hr). The peak flow from Equation (6-8) is

$$\begin{aligned} Q_p &= 0.00278 \cdot 0.37 \cdot 142 \cdot 40 \\ &= 5.8 \text{ m}^3/\text{s}. \end{aligned}$$

RATIONAL METHOD DESIGN WITH KINEMATIC WAVE

EXAMPLE 6-6

Drainage design is to be accomplished for a 4-ac asphalt parking lot in Tallahassee for a 5-yr return period. The dimensions are such that the overland flow length is 1000 ft down a 1% slope.

- (a) What will be the peak runoff rate?

A runoff coefficient of 0.95 will be assumed (see Table 6-5), as will a Manning roughness coefficient of 0.013 for asphalt (Chow, 1959, and Table 4-2). From Example 6-2, the kinematic wave parameters for this problem are $\alpha = 11.46 \text{ ft}^{1/3}/\text{s}$ and $m = 5/3$ for turbulent flow. The IDF curves of Figure 6-5 may be used to obtain intensity i as a function of duration t_r . However, an iterative procedure must be used, since the time of concentration t_c is also a function of i .

Two options for the iterative procedure have been explained in the text. Use of the Meyer equation to approximate an IDF curve is a good alternative if extrapolation of the IDF curve is required, as could occur if the necessary time were less than the minimum value of 8 min given in Figure 6-5. Alternatively, values can simply be read from the graph and inserted into Equation (6-6) on a trial basis until $t_r = t_c$. This second method is used here and is easily facilitated using a spreadsheet program, as indicated in Table E6-6.

Table E6-6 Iterative Calculations for Rational Method

Trial t_r (min)	Intensity i i (in./hr)	Calculation of	
		t_c (min)	$Q_p = CiA$ (cfs)
10	6.4	8.45	24.3
8	6.6	8.35	25.1
8.4	6.5	8.40	24.7

For the calculations, a value of t_r is assumed and a value of i is read from Figure 6-5. Rainfall excess is computed by multiplying intensity from the IDF curve by the runoff coefficient, followed by computation of t_c and Q_p . (These last three steps are done automatically by the spreadsheet program.) Although only the final value of Q_p is of use, the last column shows the relative insensitivity of the peak flow to the changes in t_r due to the “flatness” of the IDF curve in this region. Caution must be taken to use consistent units in Equation (6-6)! Thus, when evaluating the equation, rainfall intensity must be converted to ft/s and the resulting value of t_c converted from seconds to minutes. For example, the first trial for $t_r = 10$ min gives

$$t_c = \frac{\left[\frac{1000}{11.46 \cdot (6.4 \cdot 0.95 / 43,200)^{2/3}} \right]^{0.6}}{60} = 8.45 \text{ min,}$$

where the factor of 43,200 converts in./hr to ft/s. Because of the “flat” response of t_c , convergence is easily obtained in this example, to the accuracy of the IDF curves. (Remember that the last two columns are automatically recalculated by the spreadsheet software.)

- (b) What size concrete pipe ($n = 0.015$) is required for drainage at the downstream end of the parking lot, for an available slope of 2%?

Standard pipe sizes (in the United States) come in 6-in. increments to a 60-in. diameter, followed by 12-in. increments. Trial computations using Manning’s equation show that a 2-ft-diameter pipe is adequate. The full-flow capacity (about 8% less than maximum capacity) is

$$\begin{aligned} Q_f &= \left(\frac{1.49}{n} \right) A_f R^{2/3} S^{1/2} \\ &= \left(\frac{1.49}{0.015} \right) 3.14 \cdot 0.5^{2/3} \cdot 0.02^{1/2} = 27.8 \text{ cfs.} \end{aligned}$$

- (c) For the calculation of the time of concentration, should a correction be made for the wave travel time in the drainage pipe?

The length of the pipe is equal to half the width of the 4-ac parking lot. The width is

$$W = 4 \text{ ac} \times 43,560 \text{ ft}^2/\text{ac} / 1000 \text{ ft} = 174 \text{ ft,}$$

so the half-width is 87 ft. During the approach to equilibrium, let average conditions in the pipe be represented by a half-full state. Using Manning’s equation, we find the velocity

$$V = \left(\frac{1.49}{0.015} \right) 0.5^{2/3} \cdot 0.02^{1/2} = 8.85 \text{ ft/s.}$$

The wave speed is, from Equation (6-7),

$$c = 8.85 + \left(\frac{32.2 \times 1.57}{2} \right)^{1/2} = 13.9 \text{ ft/s},$$

where the half-full area is 1.57 ft^2 and the half-full top width is equal to the diameter of 2 ft.

The wave travel time is $87/13.9 = 6.3 \text{ s}$. If added to the overland flow value for t_c , this would amount to such a small correction that it could not be seen while working with the graphical IDF curves. Hence, this correction can be neglected.

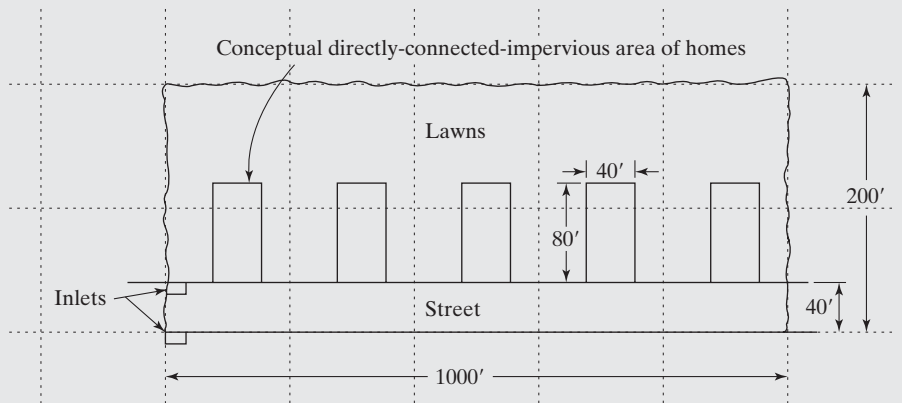
In large catchments, the drainage channels may also need to be considered in the estimate of t_c . For this purpose, the wave travel times along the flow pathways may be added to yield an overall time of concentration (see the SWMM5 example at the end of this chapter). Travel times in channels and conduits should be based on the wave speed given by Equation (6-7), using an estimate of the channel size, depth, and velocity. Since one objective may be the design of this channel, it may again be necessary to iterate to find the matching values of t_c and t_r . This possibility is examined in Example 6-6. However, in most cases, the overland flow wave travel time dominates the t_c computation.

Directly connected impervious area (DCIA) or other land uses with a high runoff coefficient that are part of a larger catchment may contribute a higher peak flow by themselves than will the whole catchment. This is because the combination of shorter time of concentration and higher C -value may overcome the smaller tributary area (Water Environment Federation and American Society of Civil Engineers, 1992). Thus, two calculations are often necessary to see which yields the higher peak flow, as illustrated in Example 6-7.

RATIONAL METHOD DESIGN WITH SUBAREA CHECKS

EXAMPLE 6-7

A Tallahassee subdivision contributes drainage from the segment shown in Figure E6-7. Each of the five homes has a DCIA of 3200 ft^2 , which drains to the street. The street drains to the two inlets indicated. Lawns are on “sandy soil of average slope.” A 10-yr design standard is appropriate for residential subdivisions and streets. Runoff coefficients should be taken at the “high” end of ranges. Inlet times are 8 min for the impervious area of the subdivision and 20 min for the total area (impervious plus lawns). What is the design flow at the inlets (treated as one inlet)?

**Figure E6-7**

Conceptual sketch of subdivision illustrating DCIA.

The area of the street is $1000 \times 40 = 40,000 \text{ ft}^2$. The area of the five homes is $5 \times 40 \times 80 = 16,000 \text{ ft}^2$. Hence, the total DCIA = $56,000 \text{ ft}^2 = 1.29 \text{ ac}$. The area of the total subdivision is $200 \times 1000 \text{ ft} = 200,000 \text{ ft}^2 = 4.59 \text{ ac}$. Hence, the area of the lawns is $4.59 - 1.29 = 3.30 \text{ ac}$.

From Table 6-5, the runoff coefficient for the DCIA subarea is 0.95. The runoff coefficient for the lawns is 0.15. An average runoff coefficient for the entire area is

$$\bar{C} = \frac{1.29 \times 0.95 + 3.30 \times 0.15}{4.59} = 0.37.$$

From Figure 6-5, the 10-yr, 8-min rainfall intensity for Tallahassee is 7.5 in./hr, and the 10-yr, 20-min intensity is 5.7 in./hr. Hence, for the DCIA only,

$$Q = C_{\text{DCIA}} i A_{\text{DCIA}} = 0.95 \times 7.5 \times 1.29 = 9.2 \text{ cfs.}$$

For the total area,

$$Q = \bar{C} i A_{\text{total}} = 0.37 \times 5.7 \times 4.59 = 9.7 \text{ cfs.}$$

The total area governs in this case, and the sewer downstream of the inlet should be sized for 9.7 cfs. But there are frequent occasions when the DCIA will govern. Both areas should be routinely checked during application of the rational method. At this point, the pipe again could be sized, as in Example 6-6, part (b).

In summary, the rational method is based on the false assumption that losses may be treated as a constant fraction of rainfall regardless of the amount of rainfall or of the antecedent conditions. Fortunately, this assumption gets better as the degree of urbanization increases (i.e., as imperviousness

increases), and intelligent use of the rational method (i.e., in conjunction with the kinematic wave equation for time of concentration) should yield a reasonable approximation to the desired peak flow. A more detailed rational method design example is shown in Chapter 9.

Coefficient and Regression Methods

Regression techniques are often incorporated into other methods such as synthetic unit hydrographs (Chapter 2), in which parameters of the hydrograph are related to physical parameters of the urban catchment. Peak flows may also be found by regression analysis (e.g., Bensen, 1962), but this application is usually performed only for large nonurban catchments. The USGS also uses regional regression techniques to relate frequency-analysis results to catchment characteristics (Sections 3.7). However, the most straightforward application is to determine a relationship between measured values of desired independent and dependent variables, such as rainfall and runoff.

When measured rainfall and runoff data are available, it is common to regress the runoff against the rainfall. If a linear equation is fit to the data, it will have the form

$$R = CR(P - DS), \quad (6-11)$$

where

R = runoff depth,

P = rainfall depth,

CR = slope of the fitted line (approximate runoff coefficient),

DS = depression storage (depth).

Here depths, not flows, are predicted. Since the depths are equivalent to volumes (when multiplied by the catchment area), depth and volume are often used interchangeably. The form of Equation (6-11) assumes that there will be no runoff until the depression storage is filled by rainfall. (If DS is negative, the equation can be rewritten to indicate a positive intercept on the runoff axis. This situation implies a base flow—that is, a contribution to runoff even with zero rainfall.) The slope CR is almost equivalent to a runoff coefficient [Eq. (6-9)], except that part of the losses is incorporated into the depression storage. Equation (6-11) works best for monthly or annual totals rather than for individual storm events and is not suited for situations in which there is appreciable carryover in the catchment storage (Diskin, 1970).

REGRESSION OF RUNOFF VS. RAINFALL

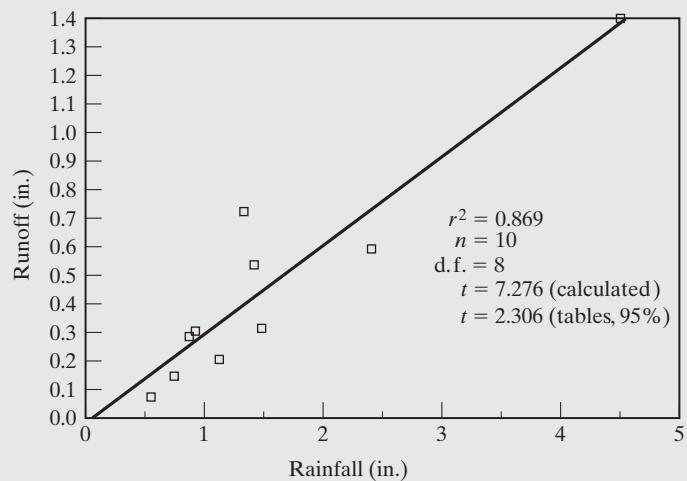
EXAMPLE 6-8

Rainfall and runoff volumes (depths) for 10 monitored storms for the Megginis Arm catchment in Tallahassee, Florida, are listed in Table E6-8 and plotted in Figure E6-8. The fit of the linear regression is also shown in the figure. The parameters listed in the figure indicate that the regression

Table E6-8 Rainfall-Runoff Data for Megginnis Arm Catchment, Tallahassee, Florida

Date of Storm Event			Rain (in.)	Runoff (in.)
Mo	Day	Yr		
6	6	79	1.48	0.316
9	21	79	1.13	0.201
9	25	79	0.54	0.070
9	26	79	2.40	0.586
9	27	79	0.73	0.148
3	9	80	0.92	0.296
4	10	80	1.35	0.721
5	22	80	0.86	0.285
5	23	80	1.43	0.537
2	10	81	4.55	1.400

Figure E6-8
Runoff depth vs.
rainfall depth for
10 storm events
in Tallahassee,
Florida.



is significant at the 95% level (Haan, 2002), although in this instance the regression is heavily influenced by the single “large” data point at 4.55 in. of rain and 1.4 in. of runoff. The fitted line has the equation

$$\text{runoff} = 0.308(\text{rainfall} - 0.059),$$

with both parameters measured in inches. Although the depression storage value of about 0.06 in. indicates that at least that much rain must fall before runoff is expected, the parameter is not significantly different from zero in this instance. This is typical of urban areas in which impervious land cover tends to generate some runoff even for small rainfall totals.

Regression—Flood Frequency Methods

The USGS extrapolates urban flood frequencies by regression analysis. Peak flow prediction by this method has been performed for urban areas for the United States as a whole (Sauer et al., 1983) and for many individual cities. For example, flood volumes and flood peaks have been analyzed for Tallahassee by Franklin (1984) and Franklin and Losey (1984), respectively, and by Laenen (1983) for Oregon's Willamette Valley. The procedure is one way of performing a *regional analysis* and is described in Section 3.7, and example regression equations for Tallahassee are given in Table 3–9.

Unit Hydrographs

Unit hydrographs, introduced in Chapter 2, have also been applied in urban areas, with the parameters of the UH related to catchment parameters by regression analysis. This was first performed by Eagleson (1962) and later by several researchers, including Brater and Sherrill (1975) and Espey et al. (1977). Applications of UHs were presented in Chapter 2 and are an option in the HEC-HMS model (Chapter 5).

Time–Area Methods

Time–area methods have also been applied with success in urban areas because of the relative ease of estimation of losses. The origin of the time–area curve was discussed in Chapter 1, but it should be emphasized that, strictly speaking, the isochrones of equal travel time shown in Figure 1–23 should be travel times of *waves*, not parcels of water, to reach the outlet, since the principle involved is one of time to equilibrium from each contributing area—i.e., the same principle discussed earlier in the context of the kinematic wave. This concept is often ignored in applications of time–area methods, but a relatively easy correction to water parcel travel times can be made using Equation (6–2), in which water velocities are multiplied by the appropriate kinematic wave exponent to approximate wave velocities. Two urban models that employ time–area methods are the Road Research Laboratory (RRL) model (Stall and Terstriep, 1972; Watkins, 1962) and the ILLUDAS model (Terstriep and Stall, 1974). The time–area method is demonstrated in Problems 6.14 and 6.15.

If the time–area diagram is approximated by an isosceles triangle, with base t_c , and outflows are then routed through a linear reservoir; the technique is sometimes called the O'Kelly model (O'Kelly, 1955; Dooge, 1973). The linear reservoir is intended to provide attenuation not represented by the translation effect of the time–area diagram. The Santa Barbara method (Stubchaer, 1975; Wanielista, 1990) consists of routing rainfall excess, weighted from impervious and pervious areas, through a linear reservoir with time constant equal to the time of concentration. The Santa Barbara method is illustrated in Problem 6.22.

Kinematic Wave Methods

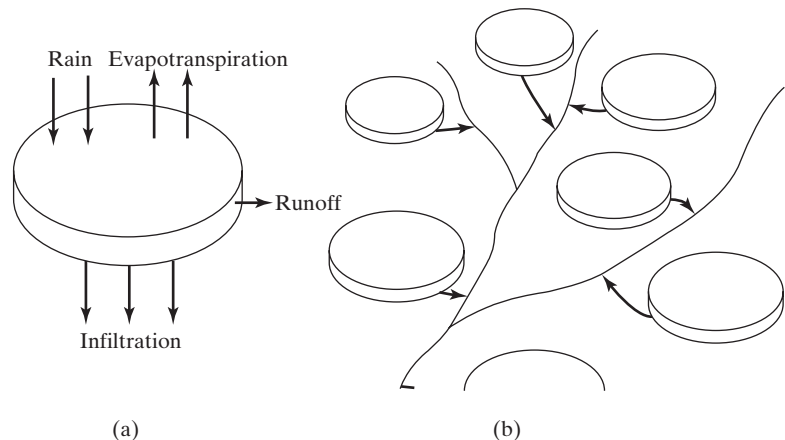
The kinematic wave equations have been discussed earlier in this chapter as well as in Chapter 4. The numerical techniques shown in Chapter 4 are usually necessary for urban areas with unsteady and nonuniform rainfall excess. However, the analytical solution for the case of steady, uniform rainfall excess (Eagleson, 1970) highlights the concept of wave travel times [Eqs. (6-2) and (6-7)] and the time of concentration [Eq. (6-6)], both immensely useful in urban hydrology. An example of a watershed model that uses kinematic waves (HEC-HMS) was discussed in Chapter 5. The SWMM model (Huber and Dickinson, 1988; Rossman, 2005) optionally uses the kinematic wave for the channel routing component (see Section 4.6). A distributed model for overland flow based on the kinematic wave applied to a grid system is presented in Chapter 11.

Linear and Nonlinear Reservoirs

A catchment surface may be conceptualized as a “reservoir” with inflows due to rainfall (and possible upstream contributions) and outflows due to evaporation, infiltration, and surface runoff (Fig. 6-7(a)). This has led to several methods for conversion of rainfall into runoff, based on solutions to the reservoir routing equations (Dooge, 1973). For use of reservoir methods, surface storage is spatially lumped; that is, there is no variation with horizontal distance, and the storage is conceptualized as a “tank” with inflows and outflows. Spatial variations may be incorporated by distributing these lumped storages over the catchment to reflect parameter variations and so on (Fig. 6-7(b)). The distributed storages are then linked by channel routing routines. The equations for reservoir routing can be solved both for conversion of rainfall into runoff and for application to “real” reservoirs, such as detention basins. The latter will be described in Section 6.6.

Figure 6-7

Conceptual reservoir models.
(a) Individual catchment.
(b) River network.
(Note: “Reservoirs” are “shapeless.” A circular shape is used only for convenience.)



As shown in Chapter 4, several routing techniques are based on a lumped form of the continuity equation:

$$Q_i - Q = dV/dt, \quad (6-12)$$

where

$$\begin{aligned} Q_i &= \text{inflow (cfs)}, \\ Q &= \text{outflow (cfs)}, \\ V &= \text{storage (ft}^3\text{)}, \\ t &= \text{time (s)}. \end{aligned}$$

The inflow $Q_i(t)$ may consist of upstream flows or rainfall or both and is assumed to be known. A second equation is thus needed to solve for the two unknowns, $Q(t)$ and $V(t)$, such as a weir, orifice, or rating curve. The two equations are then solved for the two unknowns: Q and V . Storage-indication and numerical (e.g., Runge-Kutta) methods are two solution alternatives shown for this purpose in Chapter 4. The former is especially adaptable to complex rating curves formed by the combination of multiple outlets.

An additional alternative for the “second equation” is to use the relationship

$$Q = aV^b, \quad (6-13)$$

where a and b are power-function parameters that may be fit by regression techniques or through physical relationships. For example, outflow by a weir or orifice or by Manning’s equation lends itself naturally to a power function, especially if depth $h(t)$ is used as the dependent variable instead of $V(t)$, by the relationship

$$\frac{dV}{dt} = A(h) \frac{dh}{dt}, \quad (6-14)$$

where

A = surface area and is a function of depth h .

Then a weir outflow can be represented as

$$Q = C_w L_w (h - h_0)^{1.5}, \quad (6-15)$$

where

$$\begin{aligned} L_w &= \text{weir length (perpendicular to flow)}, \\ h &= \text{water surface elevation upstream of weir crest}, \\ h_0 &= \text{weir crest elevation}, \\ C_w &= \text{weir coefficient} = C_e(2/3)\sqrt{2g}, \text{ illustrating its dimensionality} \\ &\quad (\text{length}^{1/2}/\text{time}), \\ C_e &= \text{effective discharge coefficient, and} \\ g &= \text{gravitational acceleration.} \end{aligned}$$

The weir coefficient C_w is obviously dimensional and depends on several factors, especially the weir geometry (Davis, 1952; King and Brater, 1963; Daugherty et al., 1985; French, 1985). Approximate values for sharp-crested, rectangular horizontal weirs perpendicular to the flow direction are $C_w = 3.3 \text{ ft}^{0.5}/\text{s}$ for U.S. customary units and $C_w = 1.8 \text{ m}^{0.5}/\text{s}$ for metric units.

An orifice would be included as

$$Q = C_d A_0 \sqrt{2g(h - h_0)}, \quad (6-16)$$

where

- C_d = discharge coefficient,
- A_0 = area of orifice,
- h = water surface elevation,
- h_0 = elevation of orifice centerline.

Submerged culverts often behave as orifices with discharge coefficients ranging from 0.62 for a sharp-edged entrance to nearly 1.0 for a well-rounded entrance (Daugherty et al., 1985). Apart from their universal presence along highways, culverts are widely used as outlets from detention ponds in urban areas.

Finally, Manning's equation can be used as the second relationship between storage and outflow. For a wide rectangular channel (as for overland flow), the hydraulic radius is equal to the depth, and Manning's equation has the form

$$Q = W \frac{k_m}{n} (h - DS)^{5/3} S^{1/2}, \quad (6-17)$$

where

- W = width of (overland) flow,
- n = Manning's roughness,
- DS = depression storage (depth),
- S = slope.

[The constant k_m was discussed in conjunction with Eq. (6-4).] The relationship [Eq. (6-17)] can be coupled with the continuity equation for generation of overland flow, as shown in Example 6-9.

EXAMPLE 6-9

NONLINEAR RESERVOIR MODEL FOR OVERLAND FLOW

Derive a method for generation of overland flow from rainfall by coupling the continuity equation, Equation (6-12), with Manning's equation, Equation (6-17).

Let inflow to the “reservoir” equal the product of rainfall excess i_e and catchment area A . Using U.S. customary units, Manning’s equation can be substituted into the continuity equation, yielding

SOLUTION

$$i_e A - W(1.49/n)(h - DS)^{5/3} S^{1/2} = A \frac{dh}{dt}, \quad (6-18)$$

in which the surface area A is assumed to be constant. Dividing by the area gives

$$i_e + WCON(h - DS)^{5/3} = \frac{dh}{dt}, \quad (6-19)$$

where

$$WCON \equiv \frac{1.49WS^{1/2}}{An} \quad (6-20)$$

and lumps all Manning’s equation parameters into one comprehensive constant.

For use in a simulation model, Equation (6-19) must be solved numerically, since i_e can be an arbitrary function of time and since the equation is nonlinear in the dependent variable h . This is readily done using any “equation solver” for ordinary differential equations, such as the Runge–Kutta methods discussed in Chapter 4. In the SWMM5 model, a fifth-order Runge–Kutta (RK5) method is used (Rossman, 2005).

The special case of $b = 1$ in Equation (6-13) corresponds to a **linear reservoir**, which is often used as a conceptual catchment model (Zoch, 1934; Dooge, 1973) and sometimes as a reservoir or stream routing model (Chapters 2 and 4). Routing using a linear reservoir is easily accomplished using the Muskingum method shown in Chapter 4 with Muskingum parameter $x = 0$. Analytical solutions are also available that may be evaluated with more or less ease depending on the functional form of the inflow $Q_i(t)$ (Henderson, 1966; Medina et al., 1981). The general case of a nonlinear reservoir $b \neq 1$ in Equation (6-13) has received analytical attention (see Dooge, 1973, for a summary) and is readily solved by numerical integration of Equations (6-12) and (6-13).

Flow Routing

A sewer system is similar to any other network of channels except that the geometry is well known and regular (e.g., a system of circular pipes). It is often necessary, however, to conduct a field survey to obtain invert and ground surface elevations, since the system may have been altered or plans

6.5 SEWER SYSTEM HYDRAULICS

may have been lost since construction. In principle, open-channel flow through such a system should be readily described by flow routing methods that properly account for the geometry of the sewer cross section. Many shapes are found in various cities (e.g., circular, horseshoe, egg); the geometries of several are described by Chow (1959), Davis (1952), and Metcalf and Eddy (1914). (Descriptions of some shapes can be found only in very old references!) Flow routing is often made more complicated, however, by energy losses at various structures and inflow points, possible surcharging and pressure flow regimes, supercritical flow and transitions in steep areas, and complex behavior at diversions such as pump stations and combined sewer overflow regulators (Fig. 6–2). Mays (2001b) provides a good summary. The energy grade line along a hypothetical sewer with surcharge conditions is shown in Figure 6–8 (see also Section 9.2).

For a network of pipes with no complex structures, flow routing can be accomplished by almost any of the methods discussed in Chapter 4. For example, kinematic wave, Muskingum, nonlinear reservoir, and diffusion methods have all been applied to urban drainage systems, but mostly in the context of urban runoff models, since numerical solutions are almost always required. A unit hydrograph is an example of a “combination method,” which combines the runoff generation and routing.

Saint Venant Equations

The “simpler” methods previously discussed in Chapter 4 (those that use fewer than all the terms in the momentum equation) have the advantage of computational simplicity but are limited to (1) situations in which backwater in the sewer network is unimportant and (2) dendritic networks, in which any branching of the network in the downstream direction does not depend on flow conditions downstream. Weirs and combined sewer regulators satisfy

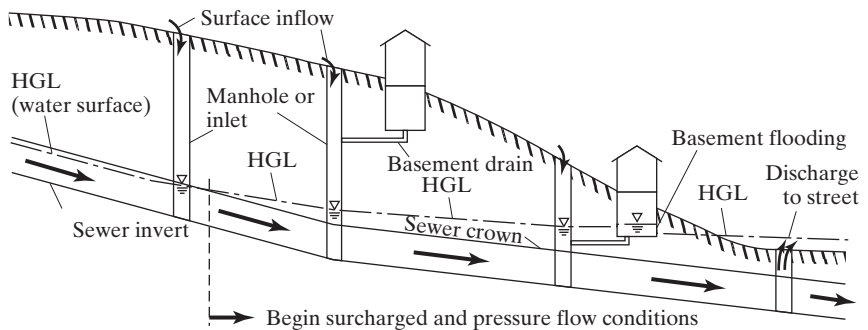


Figure 6–8

Hydraulic grade line (HGL) along a surcharged sewer segment. High HGL at downstream end could be caused by high tail-water elevation or excessive inflows to sewer system.

the latter condition as long as they are not submerged due to downstream backwater effects, and for many urban drainage networks backwater may be neglected through most of the system. In areas in which backwater must be considered, it is sometimes due to a “reservoir effect,” in which a horizontal water surface extends upstream in the network due to the ponding action of a weir, for instance. This situation may be treated as an in-line reservoir. But the most general case of a drainage system with low slopes and possible downstream boundary condition of a fixed water level (as at the entrance to a large river, lake, or estuary) requires the complete Saint Venant equations [Eqs. (4-23) and (4-26)] to simulate the resulting backwater effects. Solution of the complete equations is also necessary in the case of looped networks, in which the flow along each pathway of a loop is a function of downstream conditions. Note that this transient analysis differs from the classical steady-state backwater analysis (illustrated in Chapter 7).

The numerical methods discussed in Chapter 4 for application to the Saint Venant equations can be applied equally well in urban drainage systems; a survey of several different approaches is given by Yen (1986) and Mays (2001b). Successful models have been built on the method of characteristics (Sevuk et al., 1973), explicit methods (Roesner et al., 1988), and the four-point implicit method (Cunge et al., 1980). However, the method of characteristics seems to have the most limitations regarding the special structures and flow conditions encountered in sewer systems; in particular, both the explicit and implicit methods are more easily adapted to surcharge conditions. Simulation models that solve the full Saint Venant equations in an urban setting include the dynamic routing option of SWMM5 (see Section 6.7; Roesner et al., 1988; Rossman, 2006), the UNET model of the Corps of Engineers, Hydrologic Engineering Center (included with HEC-RAS—see Chapter 7), the FEQ (“full equations”) model of the USGS, as well as several American and European proprietary models. All models just discussed are also capable of simulating the fully dynamic regime in natural channels, in the manner of FLDWAV (Chapter 4), and URLs for each are given in Appendix E.

Surcharging

Surcharging occurs when the hydraulic grade line (HGL) of the flow is above the sewer crown, as in the downstream segment sketched in Figure 6–8. If the HGL rises above the level of basement drains connected to the sewer system, water will be backed into basements and will cause basement flooding. Health problems may also result if combined sewage enters basements, as sometimes occurs in older cities. The most spectacular effects of surcharging are blown manhole covers and geysers out of the sewer system when the HGL rises well above the ground surface. If the HGL remains below the ground surface and below the level of basement drains, usually no special problem is associated with surcharging, and the flow may even be accelerated by pressurizing the conduit.

Since surcharged conduits are really just a network flowing under pressurized conditions, one way of simulating the surcharge conditions is to treat the network like a water distribution network, in which an iterative solution is used for heads (HGL) at nodes while maintaining continuity and friction losses. This method has been applied on a steady-state basis by Wood and Heitzman (1983) and as part of the more general SWMM5 Dynamic Routing procedure (Roesner et al., 1988; Rossman, 2006). Another option is to use the Preissmann slot method, in which a small “imaginary” slot above the sewer allows free-surface flow to be maintained at all times and permits the use of only the Saint Venant equations. Models that use this scheme include Carredas (SOGREAH, 1977) and S11S (Hoff-Clausen et al., 1981).

The transition from free-surface flow to a surcharged condition is not necessarily smooth and may be accompanied by the release of trapped air, hydraulic jumps, and pressure waves (McCorquodale and Hamam, 1983; Song et al., 1983). No general sewer routing model deals with this problem; special hydraulic model studies or numerical techniques are needed and must be used if it is important.

Routing at Internal Hydraulic Structures

Structures that may be found in a storm or combined sewer include weirs, orifices, pump stations, tide gates, storage tanks, and diversion structures or regulators of various kinds that may or may not behave as a conventional weir or orifice. To the extent that they do, Equation (6-16) may be used to represent an orifice and Equation (6-15) a weir. Special allowance must be made for submerged weirs and weirs of other geometries (see Fig. 9-17 or Davis, 1952; Chow, 1959; King and Brater, 1963; French, 1985; Mays, 2001b).

Side-flow weirs are often encountered in sewer systems. An exact analysis requires analysis of the gradually varied flow past the weir (Chow, 1959; Henderson, 1966; Metcalf and Eddy, 1972; Hager, 1987; Debo and Reese, 2003). Early experimental work by Engels (1921) resulted in the following equation for total discharge over a side-flow weir in a rectangular channel, for which the water surface profile rises along the length of the weir:

$$Q = 3.32L^{0.83}h^{1.67}, \quad (6-21)$$

where

Q = side-flow weir discharge, cfs,

L = weir length (in direction of main channel flow), ft,

h = head on lower (downstream) end of weir, ft.

For a more rigorous solution, the flow per unit length along the side-flow weir is

$$q = C_s h^{3/2}, \quad (6-22)$$

where

$$\begin{aligned} q &= \text{discharge per unit length over the side weir (cfs/ft or m}^2\text{/s),} \\ h &= h(x) = \text{head along the weir crest (ft or m),} \\ C_s &= \text{weir coefficient (ft}^{1/2}\text{/s or m}^{1/2}\text{/s).} \end{aligned}$$

The head will vary along the length of the weir, and a numerical solution for the head and discharge is required (Henderson, 1966; Metcalf and Eddy, 1972; Debo and Reese, 2003). The coefficient C_s is a function of the geometry of the weir installation and of the upstream Froude number. As an approximation, a constant value of 4.1 for U.S. customary units and 2.26 for metric units may be used (Ackers, 1957). The full functional relationship for C_s is presented by Hager (1987).

Similarly, a gradually varied flow analysis may be used for the exact analysis of flow over an open **grate inlet** on the street surface. Resource material for the hydraulics of flow over grate inlets on the surface of the ground is provided by Ring (1983), U.S. Department of Transportation (1984), Water Environment Federation and American Society of Civil Engineers (1992), Mays (2001b), and Debo and Reese (2003).

Tide gates, usually in the form of **flap gates**, are often installed at the outlet of a sewer system to prevent backflow of the receiving water into the sewer conduits; they open only when the head inside the conduits is higher than the head in the receiving water. As such, they are often treated as an orifice, with discharge coefficients available from the manufacturer.

Pumps are usually operated in at least two stages. The first stage is turned on when the water in the wet well rises to a certain level, and the second stage and any additional stages are started when and if the water rises to higher specified levels. The pumps may simply raise the water to a higher level for continued free-surface flow, or they may discharge through a **force main** (a pressurized conduit). Outflow for either situation will consist of pulses as the pumps alternately turn on and off. Records of pumping cycles sometimes serve as a means for approximate flow measurements, but the actual discharge depends on the head-vs.-discharge rating curve for the pumps.

Weirs and other structures often create storage in the system. Routing through such storage can be accomplished by conventional means (Chapter 4), as described next.

Several options for control of urban runoff quantity were listed in Table 6–1, of which storage in one form or another is dominant. Regulations in many states and localities now require storage for quantity or water quality control in developing urban areas, and most new highways are designed with roadside swales (low depressions, usually vegetated) for mitigation of runoff peaks and volumes. Storage and other measures identified in Table 6–1 will be examined briefly here.

6.6 CONTROL OPTIONS

Detention/Retention

Detention storage, also known as **extended detention storage**, involves detaining or slowing runoff, as in a reservoir, and then releasing it, typically over a period of from 24 to 72 hours. In **retention storage**, runoff is not released downstream and is usually removed from storage only by infiltration through a porous bottom or by evaporation. Both types of storage are very common, although designed retention becomes less practical as the size of the drainage area increases. The required retention basin volume should be based on an analysis of storm event volumes, as discussed in Section 6.3.

EXAMPLE 6-10**DETENTION BASIN SIZING**

Obtain an estimate for the size of a detention basin required to hold the runoff from a 5-yr (volume) storm for the 2230-ac Megginnis Arm catchment in Tallahassee, Florida.

SOLUTION

The regression relationship developed in Example 6-8 can be used as a very rough method for converting rainfall into runoff. The 5-yr storm event rainfall depth is approximately 8.18 in., from Table 6-4. Thus,

$$\text{runoff} = 0.308(8.18 - 0.059) = 2.50 \text{ in.} = 0.21 \text{ ft.}$$

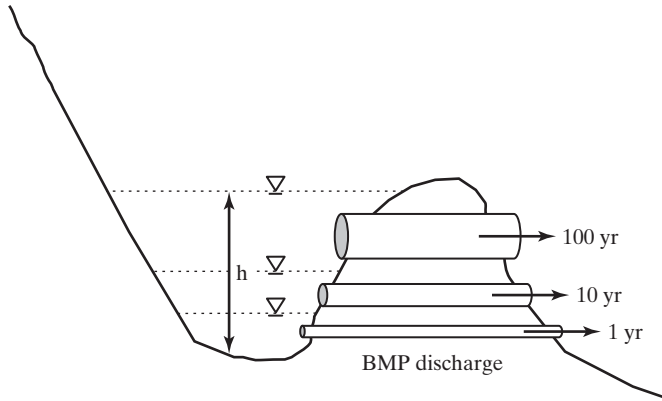
The required volume is the depth times the catchment area:

$$\text{volume} = 0.21 \text{ ft} \times 2230 \text{ ac} \times 43560 \text{ ft}^2/\text{ac} = 2.04 \times 10^7 \text{ ft}^3.$$

If the 5-yr, 24-hr depths of 7.32 in. from the Tallahassee IDF curves were used, a similar calculation would yield a runoff depth of 2.24 in. (0.186 ft) and a volume of $1.81 \times 10^7 \text{ ft}^3$. This value is slightly less than that obtained from the frequency analysis of storm event depths.

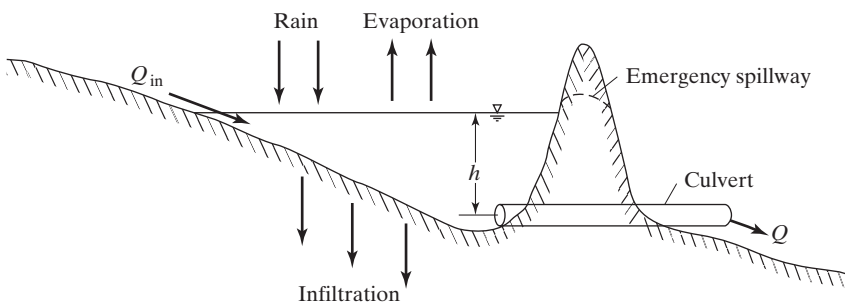
Note that this crude regression analysis ignores important factors of antecedent conditions, variable loss coefficients, and spatial and temporal rainfall variations that can be included in the analysis when using continuous simulation in rainfall-runoff models. The use of such output for detention basin sizing is demonstrated in the SWMM5 case study at the end of this chapter. Methods based on design storms are discussed by Urbanas and Stahre (1993).

Extended detention basins (also called retarding basins, or storm-water management basins) usually serve the multiple purposes of flood control and water quality control. The outlets are designed to hold a small volume of water (the “water quality volume” or “BMP volume”) for as

**Figure 6-9**

Outlet configuration for multiple-objective stormwater control.

long as possible to promote sedimentation, while providing larger outlets at higher depths to release larger inflows more rapidly (Fig. 6-9). The concept of draining within, say, 72 hours, is to have the basin be ready to receive inflow from the next storm. Continuous simulation of extended detention basins permits a design that considers the possible hold-over of water from the previous storm. A hypothetical design with a low flow outlet and emergency weir or spillway for high flows is shown in Figure 6-10. Flow routing may be performed by the methods of Chapter 4, as shown in Example 4-5. The following example illustrates how the detention pond can alternatively be treated as a nonlinear reservoir and the equations solved numerically.

**Figure 6-10**

Hypothetical detention basin.

EXAMPLE 6-11**FLOW ROUTING THROUGH A DETENTION POND**

The hypothetical design of Figure 6-10 receives as inflow the triangular hydrograph shown in Figure E6-11. Determine the outflow and stage hydrographs for the conditions given below.

Outflow is only by a culvert of 0.225 m (225 mm) diameter that behaves as an orifice [Eq. (6-16)], with discharge coefficient $C_d = 0.9$. The surface area of the basin is given by the power relationship

$$A = 400h^{0.7},$$

with A in m^2 and h in m above the culvert centerline (i.e., $h = 0$ at the culvert/orifice centerline). The basin has an initial depth of 0.5 m (above the culvert centerline).

Combining Equations (6-12), (6-14), and (6-16) yields the governing differential equation for the depth $h(t)$:

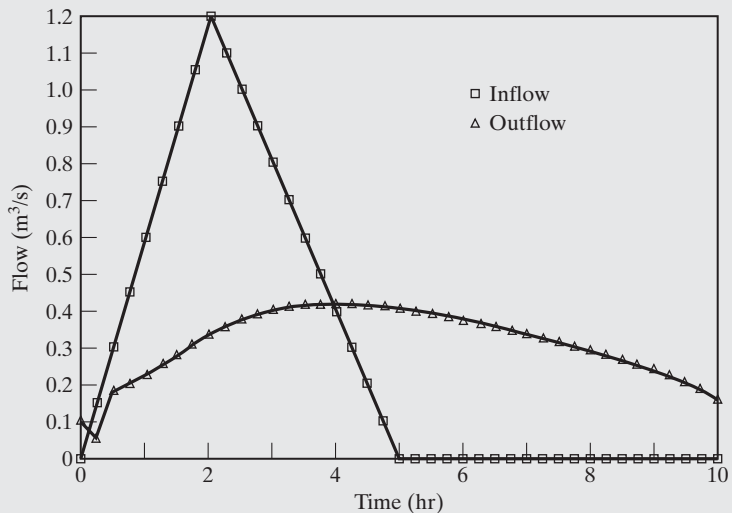
$$Q_i(t) - C_d A_0 \sqrt{2gh} = 400h^{0.7} \frac{dh}{dt}. \quad (6-23)$$

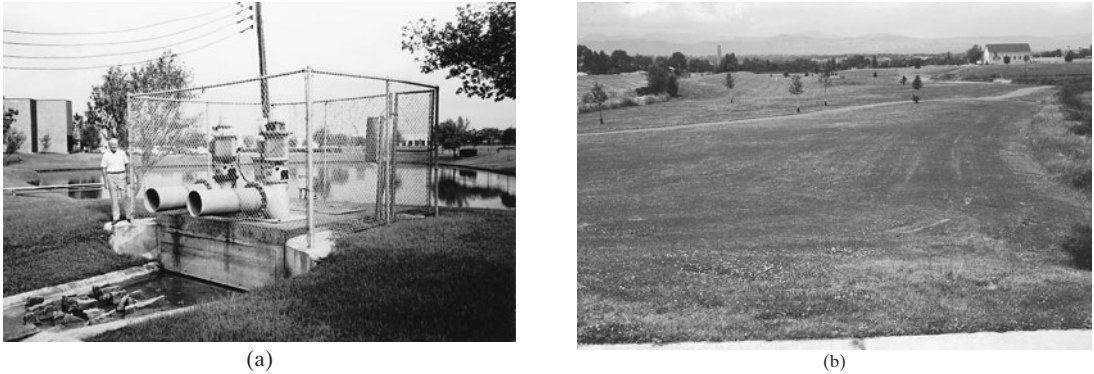
Placing the derivative on the left-hand side, dividing by the expression for surface area, and substituting for C_d and A_0 put the equation in a form suitable for numerical solution:

$$\frac{dh}{dt} = \frac{Q_i(t) - 0.0358 \cdot \sqrt{19.6h}}{400h^{0.7}}, \quad (6-24)$$

in which the substitution $g = 9.8 \text{ m/s}^2$ has also been made. Equation (6-24) may be solved by the methods already presented and demonstrated in Chapter 4, such as Euler or Runge-Kutta techniques, and its solution is not pursued further here. Results for an Euler method solution are shown in Figure E6-11.

Figure E6-11
Inflow and out-flow hydrographs for a detention pond.



**Figure 6-11**

Typical detention basin installations. (a) Storage in wet pond in Houston, with pumped outflow. (b) Large, multipurpose dry detention area in Harvard Gulch, Denver, Colorado.

Examples of detention ponds are shown in Figure 6-11. There are many practical considerations in the design of both kinds of basins, especially siting, land costs, maintenance, and the like. In some areas, the excavation may extend below the water table, resulting in a “wet pond.” A wet pond may also result if the bottom is sealed through sedimentation. Some kinds of vegetation can become nuisances in wet ponds, but wet ponds are often aesthetically pleasing as well as offering good sedimentation and pollutant removal. On the other hand, some “dry ponds” are hardly noticed by the public, since they may be implemented as multiple-use facilities, such as parks and recreation areas, and filled only during exceptional storms (Fig. 6-11b). Many details and considerations in the design and operation of detention/retention facilities and many other kinds of BMPs are given by Schueler (1987), Water Pollution Control Federation (1989), Water Environment Federation and American Society of Civil Engineers (1992, 1998), Urbonas and Stahre (1993), Horner et al. (1994), Urban Drainage and Flood Control District (1999), Mays (2001b), Washington State Department of Ecology (2001), Debo and Reese (2003), Field and Sullivan (2003), USEPA (2004), and Field et al. (2006).

In-system storage can be installed in areas where cheaper surface storage is unavailable. Such storage can consist of concrete tanks or oversized conduits, or it can simply use the sewer system volume itself, sometimes by means of adjustable weirs that are used to detain water in the system.

Surface storage may also be available on rooftops, in parking lots, and on other private lands. For example, if a roof is structurally suited, water may be ponded up to a few inches by using special drain fixtures. Rooftop vegetation for this purpose may also be employed—an example of “green engineering” (Field et al., 2000; Prince George’s County, 2000; Field and Sullivan, 2003; Field et al., 2006). Green roofs may also reduce energy costs

because of the additional insulation they provide. Unfortunately, there is little incentive for a private owner to endure problems with leaks and maintenance, and storage on private lands often is unsuccessful for this reason. (In general, maintenance of BMPs is a huge practical issue.) When storage does exist on private lands or adjacent to highways, an allowance for this “loss” must be made in the depression storage value used for the catchment.

Increased Infiltration

Since urbanization creates increased imperviousness, one likely control is to increase the amount of pervious area wherever possible. This has been accomplished with porous parking lots through the use of concrete block or similar shapes laid such that water can infiltrate through the soil-filled center. The blocks lend strength to the soil, and grass may grow on the soil so that the blocks are not visible. Monolithic permeable pavement is constructed by leaving out most of the fine fraction of aggregate of the pavement mix, and placed like regular pavement (Urbonas and Stahre, 1993), and most difficulties of possible clogging have been overcome. Interlocking porous pavement does not generally have the clogging issue, depending on the paver hole size (Field et al., 2000). Although increased infiltration is very useful for small storms and for water quality control (of small storms), it is not of much use for larger storms, for which the soil may become saturated. On the other hand, vegetated areas that facilitate infiltration also facilitate evapotranspiration (ET). Increased ET is a good way to reduce stormwater runoff volumes from more frequent small storm events, especially when BMPs for this purpose are distributed around the catchment, in the manner of low-impact development (LID). Design guidelines for several types of LID facilities are provided by Low Impact Development Center et al. (2006).

Other Methods

Inlet restrictions are sometimes used in older cities that suffer from surcharged sewer lines and for which storage is not an option. Such devices (Water Pollution Control Federation, 1989; Field et al., 2004) limit the rate at which surface runoff can enter the sewer system, thus trading off a reduction in the hydraulic grade line in the sewer for increased street surface flooding. However, the latter is usually only temporary, and this measure does reduce basement flooding. Hence inlet restrictions may be the least expensive alternative in older combined sewer systems.

Nonstructural alternatives include **floodplain zoning** and **floodproofing** of individual structures, as for general floodplain management (James and Lee, 1971; Mays, 2001a). Floodplain zoning restricts land use in flood-prone areas (often designated as the 10-yr or 100-yr floodplain) to compatible uses, such as parks, agriculture, and so on. Although some damages may still occur during flooding, major damage to the urban infrastructure is usually avoided.

Floodproofing involves raising the foundation of structures above a designated flood elevation, using either fill or pilings, or “stilts.” The methods may be used in combination, with zoning near the channel and floodproofing farther away (James and Lee, 1971). These two nonstructural alternatives are often not feasible in older cities because of the impossibility of relocating existing structures. Many additional nonstructural alternatives exist for management of stormwater quality (Field et al., 2006). These topics are covered in more detail in Chapter 12.

Definition

The evolution of hydrologic models, as discussed in Chapter 1, is a natural consequence of computer evolution and availability, theoretical developments, and data collection efforts during the past four decades. Computers and models in fact have revolutionized the manner in which hydrologic computations are now routinely performed. However, many more models have been developed than can be summarized briefly in a text, and a screening method must be employed, leading to the concept of operational models.

In this text, the term *operational* implies three conditions: (1) documentation, especially a user’s manual for operation of the program, if any; (2) use and testing by someone other than the model developer; and (3) maintenance and user support. Maintenance most often means support by a government agency; notable examples are the ACOE, Hydrologic Engineering Center in Davis, California, and the various programs within the Environmental Protection Agency. Operational models are generally available and can be acquired and successfully implemented by new users, who would have much more trouble attempting to follow a procedure documented only in a technical paper.

Examples of several operational models are given in Table 5–4. This section discusses a few of the most widely used operational models in urban hydrology that are in the public domain. Reviews and comparisons are available from Donigian et al. (1995), Singh (1995), National Research Council (2000), Mays (2001b), and Singh and Frevert (2005). Because model capabilities change rapidly, readers are encouraged to find current information from World Wide Web sites of the various model developers. Water quality processes are not reviewed in this text; a summary of methods is given by Huber (1985). Additionally, the degree of “user friendliness” varies widely from model to model. Finally, it should be noted that various American and European proprietary models offer sophistication and a good graphical user interface and might logically be investigated as alternatives to the public-domain models listed in this section. Some proprietary models also offer two-dimensional simulation of land-surface runoff with the aid of contemporary GIS and LiDAR resources.

**6.7
OPERATIONAL
COMPUTER
MODELS**

Federal Highway Administration Models

Numerous programs are available from the Federal Highway Administration for various hydrologic and hydraulic analyses related to highways. The programs may be downloaded from the website <http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm>. They include programs for water surface profiles (WSPRO, similar to HEC-RAS), culvert analysis (HY8), and general surface drainage (HY22 and HYDRAIN).

HEC-HMS

The Hydrologic Modeling System (HEC-HMS) from the U.S. Army COE Hydrologic Engineering Center has already been discussed extensively in Chapter 5. The model is designed to simulate the precipitation–runoff processes of dendritic watershed systems and includes several user options for this purpose, such as NRCS methods, unit hydrographs, and kinematic wave as well as options for continuous simulation and parameter estimation. HEC-HMS is freely available from the HEC website <http://www.hec.usace.army.mil/>.

HSPF

The first widely used continuous simulation model in hydrology was the Stanford Watershed Model (Crawford and Linsley, 1966), which was first applied in watersheds in northern California. Input precipitation data for continuous simulation included long-term hourly values from the NCDC. These hydrologic roots are incorporated into the Hydrological Simulation Program—FORTRAN (HSPF), a continuous or single-event hydrologic model that can also be used to simulate nonpoint source water quality processes (Bicknell et al., 1997). Simulation output includes time histories of the quantity and quality of water transported over the land surface and through various soil zones, down to the ground water aquifer. Although most often applied in rural situations, HSPF is readily applicable to urban applications through its impervious land module. HSPF is also a part of the EPA BASINS (Better Assessment Science Integrating Point and Nonpoint Sources) methodology for nonpoint source evaluation (Lahlou et al., 1998), in which HSPF is incorporated with a GIS for integration of spatial data with modeling capabilities. The newest version of HSPF includes a graphical user interface (GUI) and is supported by the USGS (http://water.usgs.gov/software/surface_water.html) and by the EPA Center for Exposure Assessment Modeling at Athens, Georgia (<http://www.epa.gov/ceampubl/swater/>). The model and documentation may be downloaded from the USGS or CEAM websites (Bicknell et al., 1997).

Natural Resources Conservation Service

The basics of the Natural Resources Conservation Service (NRCS, formerly the Soil Conservation Service or SCS) method (1964) have been presented in Chapter 2. The “method” is actually a thick handbook dealing with many

aspects of hydrology, from rainfall analysis to flood routing, but its best-known component uses information on soil storage, as characterized by curve numbers, to predict the runoff volume resulting from rainfall. A variety of unit hydrograph procedures are then available to distribute the runoff in time. The computerized version of the NRCS procedures is called TR-20. SCS techniques are incorporated into many programs for watershed analysis, including HEC-HMS (Chapter 5).

An adaptation of the original method to urban areas is called TR-55 (Soil Conservation Service, 1986). Additional curve numbers are provided in TR-55 as a function of urban land use, and information is presented on travel times and times of concentration for application of the unit hydrograph procedures. The SCS procedures are widely used in engineering practice, and additional source material is available (McCuen, 1982; Viessman and Lewis, 1996). The newest computerized versions of TR-20 and TR-55 are called WinTR-20 and WinTR-55, which, along with other useful hydrologic software, may be downloaded from the NRCS; search for “NRCS WinTR-55.” The strength of the NRCS method is the experience of four decades of model development and use and the enormous database of soils information; highly site-specific data can be obtained from county soils maps and NRCS Soil Survey Interpretation Sheets. (These data are also very useful in applications using methods other than the SCS method.) The weakness of the NRCS method is its synthetic design storm philosophy, which was critiqued in Section 6.3. With this method, it is also difficult to duplicate measured hydrographs in areas with high water tables. This difficulty arises from the SCS method’s sensitivity to the depth of the water table and its inability to represent this factor using the three available antecedent conditions (Capece et al., 1984).

SWMM

The Storm Water Management Model (SWMM) was developed for the Environmental Protection Agency in 1969–1971 as a single-event model for simulation of quantity and quality processes in combined sewer systems (Metcalf and Eddy et al., 1971; Huber et al., 2005). It has since been applied to virtually every aspect of urban drainage, from routine drainage design to sophisticated hydraulic analysis to nonpoint-source runoff quality studies, using both single-event and continuous simulation. By subdividing large catchments and routing flows down the drainage system, SWMM can be applied to catchments of almost any size, from parking lots to subdivisions to cities. An extensive literature exists for SWMM, exemplified in proceedings of the type edited by James et al. (2006).

Although Version 4 of SWMM (Huber and Dickinson, 1988; Roesner et al., 1988) is still useful for some purposes, most SWMM users will be best served by Version 5: SWMM5, released in 2004. This version has been carefully recoded in the C language to eliminate errors and add features, and it

includes a GUI. The model, code, and documentation (Rossman, 2005) may be freely downloaded at <http://www.epa.gov/nrmrl/wswrd/wq/models/swmm/>. Notable in the documentation is an applications manual (Gironás et al., 2009).

Runoff is generated from rainfall in SWMM using a nonlinear reservoir method (Example 6–9). Subsurface flow routing of water infiltrated through the soil surface is optional. Flow routing may be performed by the kinematic wave method or by using the complete Saint Venant equations. This dynamic wave routing remains one of the most powerful features of SWMM. Simulation of storage and hydraulic controls may be performed throughout the catchment and drainage systems. Water quality generation and transport may also be simulated throughout the catchment and drainage systems, as well as treatment in storage devices and elsewhere for modeling of most best management practices (BMPs). SWMM5 includes many graphical and other reporting tools. For instance, the statistics tool may be used to separate continuous simulation hydrographs and pollutographs (concentration vs. time) into independent storm events (as in Example 6–3), calculate statistical moments, and perform elementary frequency analyses. Metric units are optional. A detailed case study of the use of SWMM5 is given later in this chapter.

U.S. Geological Survey Models

The USGS is the leading federal agency for research in hydrology and has produced several rainfall–runoff models and flow routing software (e.g., the FEQ model described earlier), all of which are freely available from the USGS website: http://water.usgs.gov/software/surface_water.html. However, support for USGS surface water simulation tools is not as readily available as for the models previously described. In addition to simulation models, a large number of software packages for statistical analysis, sediment transport, scour analysis, and other hydrologic and hydraulic computations are also available from their generous website.

Model Selection

Although there may be theoretical reasons for preference of one model over another, the choice is often made on much more pragmatic grounds. Considerations include designation of a required model by a permitting agency, model support and documentation, familiarity with techniques used in a model, type of computer required, and availability of data. The last factor is probably the most important. For example, complex flow routing in a sewer cannot be performed without detailed information on invert elevations, ground surface elevations, and hydraulic structures in the sewer system. If such information is not available, a lumped parameter approach using a much simpler method (such as a UH) might be appropriate.

When both rainfall and runoff data are available for calibration, most of the models discussed in this section can be calibrated for the catchment

under consideration. In general, the simplest model that is applicable to the problems under study and that can be properly calibrated is probably the best choice. However, simple models are generally able to address fewer hypothetical “what-if” questions dealing with alternative design formulations, because the required physically based procedures may not be incorporated into the models. Hence model choice is an important function of model capability as well as model simplicity.

A sensitivity analysis of a model will reveal information on the relative importance of the many input parameters as well as uncertainty in model output (James and Kuch, 1998). This is an important introductory step for the use of any model.

Finally, model reviews and comparative studies referenced earlier in this section may be consulted for guidance. Since model capabilities continually change and improve, caution should be exercised when reading about models in older references and in this text. World Wide Web sites for all models are the best way to survey current model capabilities and availability.

Introduction

How may an urban rainfall–runoff model be applied to a real catchment? What kind of information results, and for what purpose? To illustrate the use of such a model, the calibration and verification of the SWMM5 model on a high-density residential (apartment) catchment, abbreviated HDR, in Miami, Florida, will be shown. Results of continuous simulations will be illustrated for purposes of frequency analysis and selection of design storms from the historic record. This particular catchment is one of four south Florida catchments monitored by the USGS (Matraw et al., 1978; Hardee et al., 1978; Miller et al., 1979; Hardee et al., 1979), which collectively form one of the best model-testing opportunities of any extant dataset. The usefulness of the measured rainfall, runoff, and quality data is tremendously enhanced by the foresight of Miller (1979) in providing data needed for modeling for each of the four catchments.

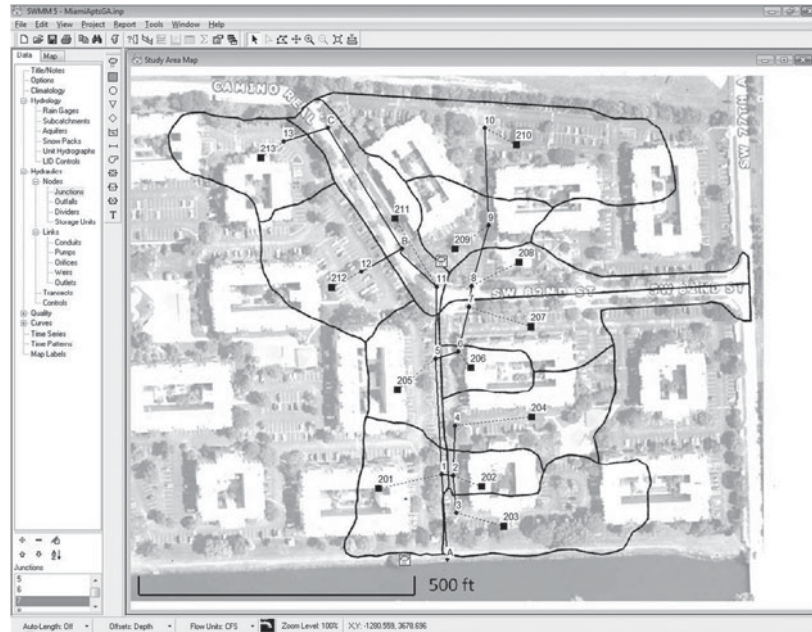
The Catchment

The 14.7-ac HDR Catchment (Fig. 6–12) discharges to Snapper Creek (South Florida Water Management District Canal C-2) in south Miami, just southwest of the interchange of state highways 826 and 878. A watershed map used in SWMM5 is shown in Figure 6–12. Hardee et al. (1979) monitored 16 storms for rainfall, runoff, and water quality between May 1977 and June 1978. The apartment complex in 2011 is essentially the same as it was during the monitoring period, although cultural changes over 30 years would be expected to change runoff quality characteristics. The HDR catchment

6.8

CASE STUDY

Figure 6-12
SWMM5 watershed area with inlets and pipes shown, Miami High Density Residential catchment.



has been studied and modeled by several investigators, including Cunningham and Huber (1987), Huber et al. (1991), Hamid (1995), and Lee and Heaney (2005). Catchment characteristics compiled by Miller (1979) are listed in Table 6-6, along with SWMM5 roughness, depression storage, and infiltration parameters and their basis.

Data Input and SWMM Calibration

Rainfall is converted into runoff in SWMM5 by the nonlinear reservoir method illustrated in Example 6-9; corresponding subcatchment input parameters are listed in Table 6-7, in addition to the global roughness, depression storage, and infiltration parameters listed in Table 6-6. The average subcatchment slope could be found by selecting several points at the edge of the subcatchment, calculating the path length of each point to the subcatchment outlet, dividing the path lengths by the change in elevation, and taking an area-weighted average of the eight slopes. In this case, average slope was computed using an Arc/Info utility TIN (triangular irregular network) applied to a digitized map of the basin (Huber et al., 1991; see also Chapter 10). Percent imperviousness is DCIA (pavement, sidewalks) from Miller's (1979) data. An additional SWMM5 option would be to enter total imperviousness and then route the non-directly connected impervious runoff ("runon") over the adjacent pervious area (the remaining DCIA would drain directly to the subcatchment outlet). This refinement was not used for this example but would be especially useful in situations for which saturation of the pervious area is likely due to runon.

Table 6-6 Hydrologic Characteristics of Miami High-Density Residential (HDR) Catchment (Miller, 1979).

Parameter	Value
Latitude, Longitude	25°40'31" N, 80°19'11" W
Area	14.7 ac (5.95 ha)
Percent total imperviousness	71
Percent directly connected imperviousness (DCIA)—excludes apartment roofs; downspouts are connected to lawn	44
Mean annual rainfall	56 in. (1420 mm)
Soil description	Perrine marl: friable marl of silt-loam texture; contains few shell fragments; underlain by limestone; NRCS hydrologic soil group D.
Streets	Asphalt streets have no curb and gutter and drain to depressed centerline of street.
Storm sewers	Circular, corrugated metal except for lowest two pipes, connecting links 5-1 and 1-A in Fig. 6-12.
Manning's roughness	
Impervious sidewalks and asphalt pavement (Table 4-2)	0.014
Pervious lawn (Table 4-2)	0.2
Corrugated metal pipe (CMP) (Table 7-1)	0.024
Reinforced concrete pipe (RCP) (Table 7-1)	0.013
Depression storage (Table 6-3)	
Impervious	0.05 in. (1.3 mm)
Pervious	0.15 in. (3.8 mm)
Green-Ampt parameters (Table 2-4)	
Suction	6.6 in. (168 mm)
Hydraulic conductivity	0.25 in./hr (6.4 mm/hr)
Initial moisture deficit	0.1

Table 6-7 Subcatchment Characteristics for HDR Catchment

Subcatchment	Area (ac)	Percent Total Imperviousness	Percent DCIA	Width (ft)	Slope (percent)
201	1.157	60	36	254	3.1
202	0.352	88	31	87	3.5
203	1.412	56	40	148	2.9
204	1.236	69	37	150	2.0
205	0.842	78	47	241	2.7
206	0.395	77	32	88	3.0
207	1.204	74	49	81	1.8
208	1.006	69	51	65	2.0
209	0.761	76	32	126	3.1
210	2.798	79	49	175	2.1
211	1.049	50	36	89	1.4
212	1.452	80	60	112	1.3
213	1.079	73	41	145	2.8

Subcatchment width is a shape parameter, often used for calibration of peak flows. It can be estimated by determining the “average maximum” flow path length to the outlet, and dividing the area by this path length. The “average maximum” is the average of several flow path lengths from the outlet to the subcatchment boundary. This is the method used by Huber et al. (1991) to estimate widths for the HDR subcatchments. However, as perhaps more of an ambiguous input parameter than slope or roughness (neither of which is necessarily straightforward to estimate), width is often used as the calibration parameter of choice for peak flows, since from Equation (6-20) it can be seen than any two of the three (width, slope, roughness) may be fixed and the third varied for calibration. Higher width usually means higher hydrograph peaks, because water runs off the subcatchment faster. However, in this modeling example, a priori input parameter estimates were sufficient to produce good calibration and verification simulations, and parameters were not varied beyond what is shown in Tables 6-6 and 6-7.

Average monthly evaporation was not included in simulations for individual storm events, since it has minimal effect over a period of only a few hours. For continuous simulation runs, average monthly pan evaporation data were obtained from NWS values published by Farnsworth and Thompson (1982) for the nearest station to the HDR catchment: Hialeah (NWS station 08309), about 14 mi north, from which actual evapotranspiration (ET) estimates were calculated by multiplying by a pan coefficient of 0.8 (Table 1-5) to reflect total ET. These ET estimates are shown in Table 6-8.

Five-minute interval rainfall-runoff data for calibration and verification were monitored and reported by Hardee et al. (1979) for 16 events. Ten of the storms were selected from the two-year study and randomly divided into two sets of five storms each, one for calibration and the other for verification

Table 6-8 Evaporation Data Used for ET Estimates
(Farnsworth and Thompson, 1982)

Month	PAN (in.)	ET = PAN × 0.8 (in.)
Jan	3.81	3.05
Feb	4.42	3.54
Mar	6.12	4.90
Apr	7.26	5.81
May	7.80	6.24
Jun	7.12	5.70
Jul	7.36	5.89
Aug	7.22	5.78
Sep	5.91	4.73
Oct	5.81	4.65
Nov	4.79	3.83
Dec	3.80	3.04

Table 6–9 HDR Catchment Monitored Storms and 10 SWMM5 Simulation Results

USGS ID	Date	Rain Gage 1 (in.)	Rain Gage 2 (in.)	Monitoring Duration* (hr)	Measured Runoff Volume (in.)	Measured Runoff Peak (cfs)	Simulated** Runoff Volume (in.)	Simulated** Runoff Peak (cfs)
1	5/4/1977	2.85	—	5.67	1.983	20.68	1.588	20.58
3	5/11/1977	1.17	—	0.83	0.657	31.57	0.669	30.58
4	5/11/1977	2.08	—	2.083	1.426	26.47	1.228	20.07
5	6/1/1977	1.86	1.66	4.25	1.176	15.64		
9	7/15/1977	1.67	1.45	3.17	0.668	15.28	0.854	15.01
16	8/8/1977	—	0.53	1.33	0.217	11.91	0.215	10.41
17	8/8/1977	0.84	1	4.33	0.541	12.98	0.365	11.51
18	8/12/1977	1.50	1.49	6.083	0.9	16.26		
33	12/6/1977	0.70	0.8	2.5	0.308	15.57	0.335	10.94
38	1/19/1978	0.73	0.79	5.5	0.277	9.99		
39	2/18/1978	2.02	2.1	7.917	0.712	10.98		
40	3/3/1978	1.56	1.51	4.67	0.423	14.04		
44	4/23/1978	0.74	0.73	2.17	0.33	14.33	0.321	12.72
48	5/18/1978	0.75	0.74	3.83	0.238	7.96	0.303	6.53
50	5/26/1978	0.61	0.58	2.17	0.266	10.53		
52	6/12/1978	1.01	1.01	1.083	0.444	24.03	0.548	25.75

*Monitoring was at 5-min intervals and duration reflects the 1/12-hr intervals. The duration corresponds to the end of flow sampling. The rainfall duration was usually less by many minutes.

**Gray shading corresponds to storms used for verification while unshaded means calibration storms.

(Table 6–9). (Except for the additional effort, all 16 storms could be used.) Rainfall was monitored at one or two gages for all storms, as indicated in Table 6–9. For SWMM simulations, gage 1 (near outlet, at bottom of Fig. 6–12) was assigned to subcatchments 201–206 and gage 2 to the remainder. Runoff was measured in the outfall pipe (connecting nodes 1-A in Fig. 6–12). A 5-min time step was used for both the single-event and continuous simulation runs.

The model was first run for the five calibration storms simultaneously—that is, while maintaining the same parameter values for each (Maalel and Huber, 1984). This results in a calibration that is less than what might be achievable for an individual storm but robust for a group of storms. The idea of verification events is to test the calibration on a set of events not used to calibrate the model, and five such verification storms were then run. A common way of demonstrating multiple-storm calibration is shown in Figures 6–13 for volumes and 6–14 for peak flows. Perfect agreement would have data points fall on a 45-degree line. Although modelers are seldom so fortunate, the agreement of measured and simulated runoff volumes (as a depth) and peak flows was deemed sufficient without further parameter adjustments; that is, the model was not calibrated using the rainfall–runoff data. Calibrations are generally easier in urban catchments with high imperviousness. A listing of the storms simulated and the data used in Figures 6–13 and 6–14 is given in

Figure 6–13
Goodness of fit of
runoff volumes.

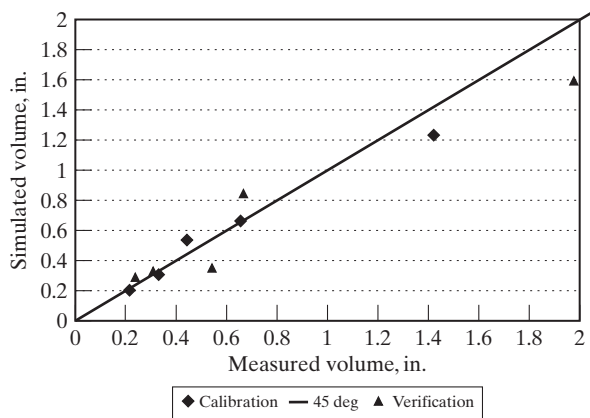
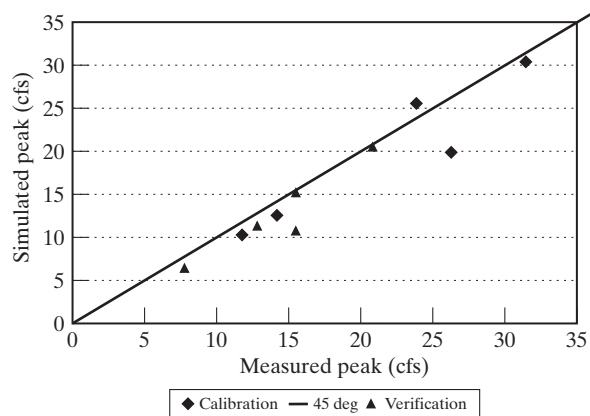


Table 6–9. Two examples of SWMM5 output graphics comparing simulated and measured hydrographs are shown in Figure 6–15. Multiobjective calibration of SWMM is discussed by Herrera et al. (2006).

Analysis of Continuous Results

When the verification was completed, a continuous run of 31 yrs (376 months: April 1972 to August 2003) was made using 15-min rainfall data from the nearest available gage with 15-min rainfall: Pennsuco, NWS 086988, approximately 18.8 miles west of the catchment. Statistical analysis of the predicted flows was performed using the Statistics Tool of SWMM5. The time series of 5-min runoff values was separated into 2199 independent storm events using an MIT value of 6 hr (see Section 6.3) and an outfall flow threshold of 0.1 cfs. The objective is to identify historic events with return periods of about 5 years (arbitrary, for this example) for possible use as design events for stormwater management.

Figure 6–14
Goodness of fit of
runoff peaks.



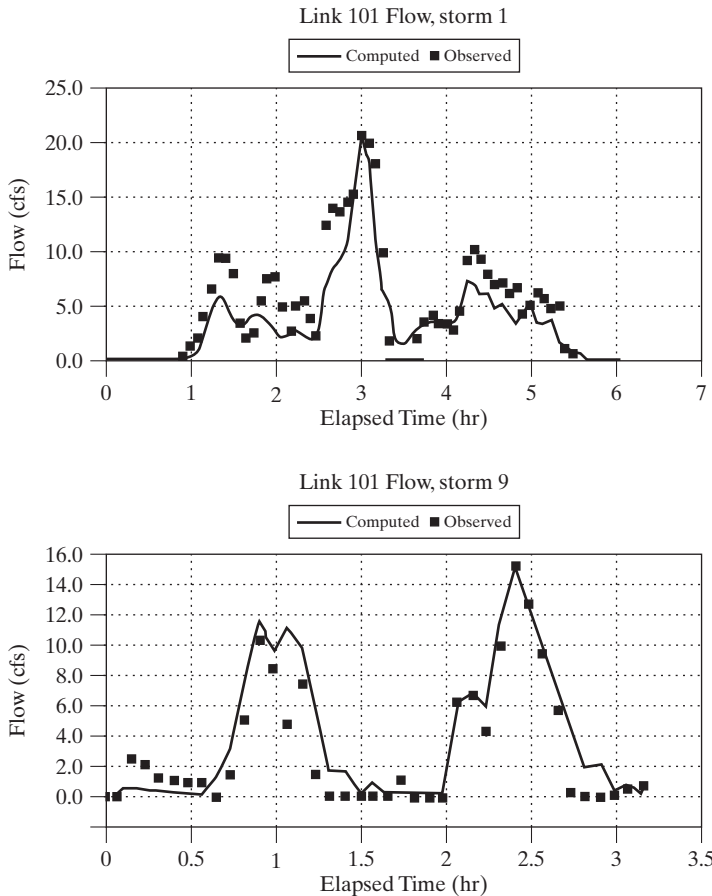


Figure 6-15
SWMM5 GUI comparison of simulated and measured flows for two storms.

The SWMM5 Statistics Tool performs a frequency analysis on any or all of the following parameters: runoff volume, average flow, peak flow, event duration, and interevent duration. (If pollutants were being simulated, frequency analyses could also be performed on total load, average load, peak load, event mean concentration, and peak concentration.) Storm events are sorted and ranked by magnitude for each parameter of interest and are assigned an empirical return period in years according to the Weibull formula [Eq. (3-75)]. The largest-magnitude event for this 31.3-yr simulation was thus assigned a return period of 32 years. For this simulation, the sixth and seventh largest events bracket a 5-yr event with return periods of 5.3 and 4.6 years, respectively, both of which are compared as “candidate design events.”

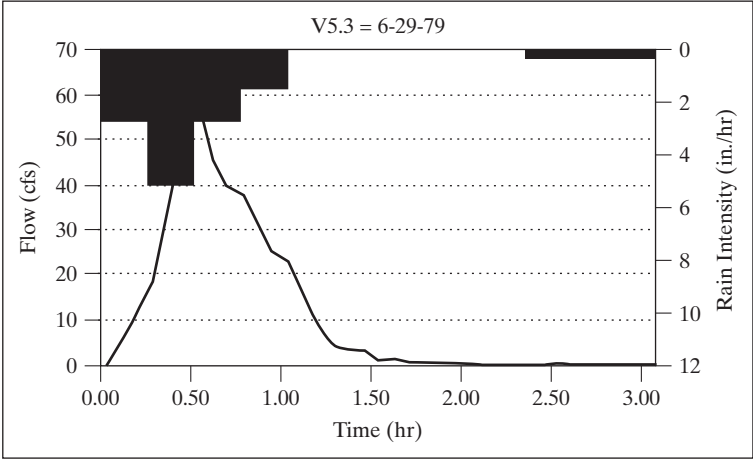
On this basis, four storms from the historical record were selected for the 5-yr storm: two based on peak flow and two based on total flow (runoff volume). The hyetographs of these storms are shown in Figures 6-16 and 6-17, and their characteristics are listed in Table 6-10. An unfortunate

Table 6-10 Characteristics of Historical Design Storms

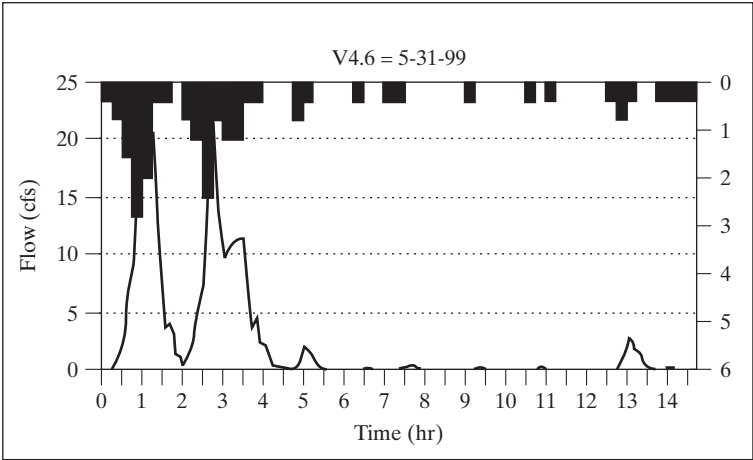
Storm Number*	Date	Runoff Duration (hr)	Runoff Volume (in.)	Peak Flow (cfs)	Rainfall Duration (hr)	Rainfall Volume (in.)	Time Since Last Event (hr)	Rainfall Last Event (in.)
V-5.3	5/31/99	14.67	2.39	23.5	13.75	5.6	66.5	0.7
V-4.6	6/29/79	2.83	2.33	57.5	2.25	3.2	23.5	0.3
P-5.3	8/20/72	2.08	1.34	49.4	1.0	2.1	47.5	0.1
P-4.6	7/15/85	2.08	2.10	48.3	1.5	3.0	31	0.7

*V means ranked by runoff volume, P means ranked by runoff peak, and numbers following are return periods in years.

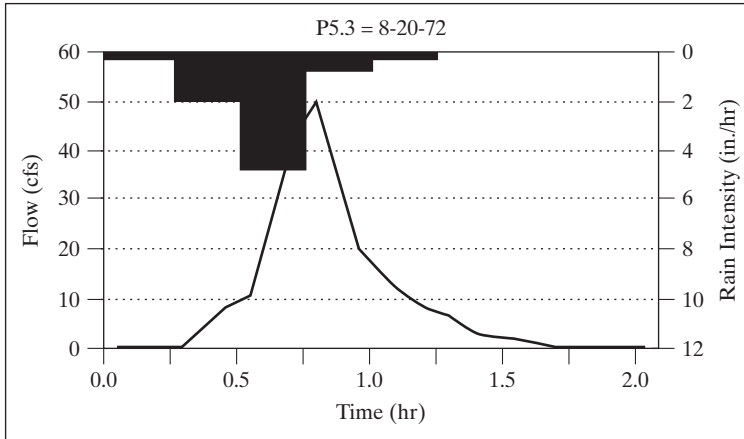
Figure 6-16
Five-year historic design events based on frequency analysis of storm event volumes.



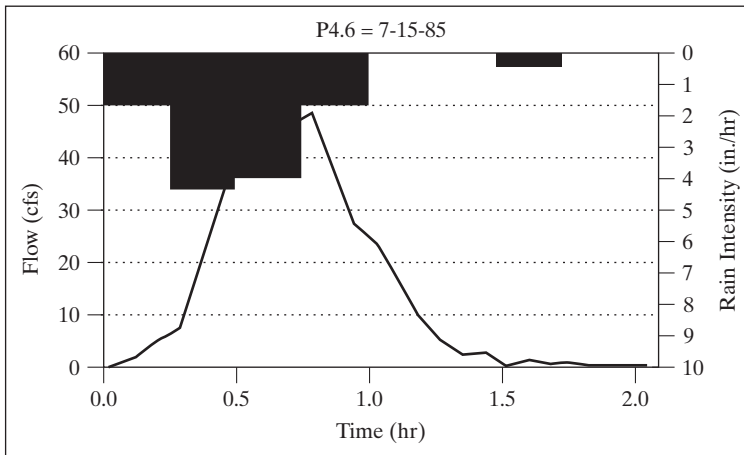
(a)



(b)



(a)



(b)

Figure 6-17

Five-year historic design events based on frequency analysis of storm event peaks.

characteristic of many of the 15-min datasets from the NCDC is apparent from the hyetographs: the minimum depth resolution is 0.1 in., instead of 0.01 in. for daily measurements and for many hourly datasets. This means that the minimum 15-min intensities will be $0.1 \text{ in.}/0.25 \text{ hr} = 0.4 \text{ in./hr}$. Small catchments such as the Miami HDR catchment can easily respond to intensities less than 0.4 in./hr , as may be seen from the calibration and verification runs (Fig. 6-15); however, shorter time resolution (15-min rainfall) data were selected over better depth resolution (hourly rainfall) data.

The modeling results emphasize the fact that the return periods of the individual storms are different when ranked by another parameter, as shown

Table 6–11 Return Periods (Years) of Storms by Volume, Peak Flow, and Rainfall

Storm Number	Date	Rank, Return Period by Volume	Rank, Return Period by Peak Flow	Rank, Return Period by Rainfall Volume
V-5.3	5/31/99	6, 5.3	75, 0.43	4, 8.0
V-4.6	6/29/79	7, 4.6	5, 6.4	29, 1.1
P-5.3	8/20/72	36, 0.89	6, 5.4	100, 0.32
P-4.6	7/15/85	10, 3.2	7, 4.6	35, 0.91

in Table 6–11. For example, storm V-5.3 is the 6th largest by volume, 4th largest by rainfall volume, but 75th largest by peak flow! On the other hand, storm V-4.6 is the 7th largest by volume, 29th largest by rainfall volume, but 5th largest by peak flow. Table 6–11 illustrates that return periods of the same event ranked by different parameters are rarely the same. Chin (2004) reports that 5-yr, 1-hr storms are used for assessing the capacity of onsite drainage structures, such as BMPs. Three of the design events shown in Figures 6–16 and 6–17 are coincidentally on the order of 1 hr duration, but could be longer or shorter, since the peaks are a result of bursts of high-intensity rain within the storm event. This points out the inconsistency of assigning an arbitrary duration to design storms, especially for a catchment this small. Continuous simulation allows the engineer to use actual durations of actual storms, avoiding the vexing problem of having to choose a duration at the start of the analysis for application of a synthetic hyetograph.

Other Design Techniques

Unfortunately, USGS regional flood frequency techniques (Chapter 3) cannot be applied to this small urban catchment in Miami because, as of 2011, no generalized equations are available for either the rural or urban areas of South Florida (<http://water.usgs.gov/software/nff.html>). However, the 5-yr rational method peak can be compared to the historic storms, and this section also serves to provide an additional rational method example. Chin (2004) provides a functional form of the IDF curves used in the Miami-Dade area:

$$i = \frac{308.5}{48.6T^{-0.11} + t(0.5895 + T^{-0.67})}, \quad (6-25)$$

where

- i = intensity (in./hr),
- T = return period (year), and
- t = rainfall duration (min).

Interestingly, for a fixed T , Equation (6–25) is of the same form as Equation (6–10). The kinematic wave overland flow travel time [Eq. (4–60)] will be

used in conjunction with assumed dynamic wave [Eq. (6-7)] travel times down the circular pipes shown in Figure 6-12. The longest travel time probably occurs from the combination of overland flow down subcatchments 207 and 208 toward inlets 7 and 8, plus the travel time down the four pipes from junction 8 to junction 1, neglecting the travel time down the outfall pipe, since that pipe typically has some standing water in it. From the SWMM5 input data, the four pipes connecting junctions 1 and 8 total 410 ft and all but one are 3 ft in diameter. The upper three pipes are corrugated metal pipe (CMP), with a Manning roughness of about 0.022 (Table 7-1), while the pipe connecting junctions 5 and 1 is reinforced concrete pipe (RCP), with a Manning roughness of about 0.013. Because the computations that follow are insensitive to most of the numbers used for the wave speed calculation, a simple average of 0.017 will be used for n instead of computation of a length-weighted harmonic mean or computation of wave speeds down the individual pipe segments. Invert elevations at junctions 8 and 1 are 2.262 and -0.889 ft NGVD, respectively, giving an average slope of 0.00769. Assuming a 3-ft diameter pipe half full, Equation (6-7) gives a wave velocity of 6.16 ft/s and a corresponding dynamic wave travel time down the four pipes of $54 \text{ s} \approx 1 \text{ min}$.

The dynamic wave travel time through the sewers will be added to the kinematic wave overland flow travel time. The latter is computed using an overland flow length of 500 ft (estimate for subcatchments 207 and 208 from Fig. 6-12), Manning roughness of 0.014 for the pavement (Table 6-6), and slope = 0.019 (SWMM input parameters for subcatchments 207 and 208). Iterations proceed as in Example 6-6, with the difference that total time of concentration = overland flow t_c + wave travel time in sewers ($\sim 1 \text{ min}$). Computations are summarized in Table 6-12.

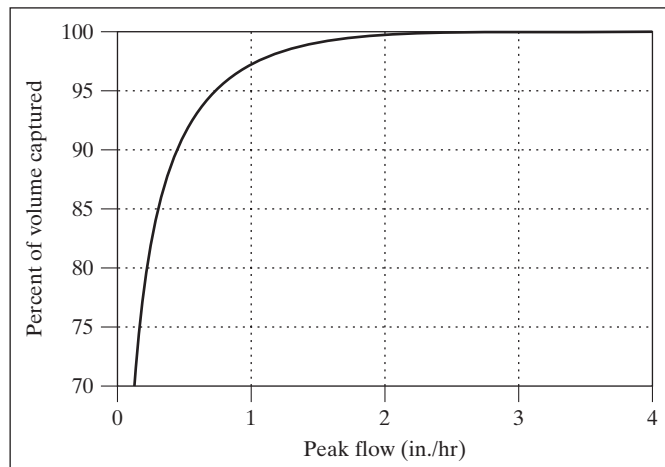
Table 6-12 Kinematic Wave Iterations for 5-yr Peak Flow

t_r (min)	i Eq. (6-28) (in./hr)	Kin. Wave t_c Eq. (4-60) (min)	t_c Total (min)	$C = 0.7$ (cfs)	$Q = CiA$ $C = 0.6$ (cfs)	$C = 0.5$ (cfs)
5	6.801	4.559	5.559			
5.6	6.718	4.582	5.582	69.1	59.3	49.4
6	6.664	4.596	5.596			
7	6.533	4.633	5.633			
8	6.407	4.669	5.669			
9	6.285	4.705	5.705			
10	6.169	4.741	5.741			
15	5.644					
30	4.497					
45	3.737					
60	3.197					

With access to a functional equation for the IDF curve, quick convergence is reached for an overall $t_c = t_r$ of about 5.6 min. Other numbers are left in the table to illustrate the relative insensitivity of both the intensity and t_c value to the assumed duration, t_r . If the reader performs his/her own computations using these data, the final rainfall intensity is generally insensitive to almost all the assumptions listed in the preceding paragraphs. However, the rational method $Q = CiA$ is highly sensitive to C , probably the least well-defined parameter among all that are being used! The runoff coefficient for “residential, multi-units” ranges from 0.4 to 0.75 from Table 6–5. The units for this apartment complex are detached but “large.” Picking a mid-range value for detached units gives $C = 0.5$, but a higher value might prevail (and be more conservative). However, a C value of 0.5 gives $Q = 49$ cfs and remarkably (and unusually) good agreement with the SWMM continuous simulation results, shown in Table 6–10, for which 5-yr peaks are in the range 48–50 cfs. The 4-ft CMP outfall sewer has a slope of 0.00505 (SWMM input) and roughness of 0.013 (Table 7–1). The full-flow computation using Manning’s equation [Eq. (4–30)] gives $Q = vA = 34$ cfs. Coupled with likely high tailwater in the Snapper Creek receiving water, the outfall is likely underdesigned for the 5-yr peak flow, based on both continuous simulation and rational method results. The result will be additional overland flow toward the canal during such events and/or temporary ponding of water on the land surface.

Finally, SWMM (and other model) continuous simulation output may be used for direct evaluation of the relationship between magnitude of runoff volume or peak flow and frequency of occurrence. SWMM5 output for a simple ranked evaluation (i.e., a duration curve of the type discussed in Section 3.5) of runoff peaks is shown in Figure 6–18. The resulting frequency curve can be interpreted as the percentage of annual runoff volume

Figure 6–18
Percentage of annual runoff volume passing through a control device that accepts up to indicated unit peak flow.



that would be captured—i.e., passed through a BMP—for a given maximum inflow for the BMP. For instance, for the HDR catchment, if it were desired to capture (and treat) 90% of the annual runoff by a flow-limited BMP (i.e., one with little or no storage volume), the device would need to be able to accept about 0.4 in./hr, unit runoff rate, or $0.4 \text{ in./hr} \times 14.7 \text{ ac} \approx 5.9 \text{ cfs}$. Regional frequency diagrams of this type have been constructed by Geosyntec Consultants et al. (2006). The shape of the curve in Figure 6–18 is very typical of an analysis of this type: Most of the control is managed through capture of “small” storm events, whether the control is based on peak flows or runoff volumes, as demonstrated by several authors, such as Heaney et al. (1977), Urbonas and Stahre (1993), and Water Environment Federation and American Society of Civil Engineers (1998). Similar analysis of volume controls must be done by simulating the storage device as well as the hydrology, to account for the draw-down between storms and the interaction between water remaining in storage and storm interevent times. Regional curves for this purpose are also provided by Geosyntec Consultants et al. (2006).

Case Study Summary

The SWMM5 model was calibrated and verified for a high-density residential apartment complex in Miami. Based on a continuous simulation, historical storm events were selected for prediction of peak flows and runoff volumes at desired return periods. A comparison was made with the 5-yr rational method peak, for which the results are highly sensitive to the choice of the runoff coefficient, C , but for which the rational method Q_{peak} was within the range of continuous simulation results using $C = 0.5$. The primary advantage of the rational method and synthetic design storms is simplicity and ease of standardization by an agency. Advantages of continuous simulation using historical storms (Linsley and Crawford, 1974) include the following:

1. Simulation using historical storms implies a frequency analysis of peak flows (or runoff volumes), not a conditional frequency analysis of rainfall depths. That is, the frequency analysis is on the parameter of interest.
2. The frequency analysis of the continuous time series of flows includes all effects of antecedent conditions, whereas analysis of the synthetic design storms does not.
3. The historical storms avoid the vexing questions of storm duration, shape, and hyetograph discretization. The duration is especially critical, since peak flows may arise out of a storm that lasts a few or several hours. There is simply no basis for establishing a standard duration, such as 24 hr, for all design work, as seems to be the unfortunate tendency in many urban areas.

4. Historical storms can also be used for analysis of volumes for design of basins for detention or retention. The volume of synthetic storms is arbitrarily linked to the assumed rainfall duration. A given volume can result from infinitely many combinations of intensity and duration, with a corresponding range of return periods.
5. If historical storms, rather than “unreal” synthetic storms, are used for design, local citizens can be confident that the design will withstand a real storm that they may remember for its flooding. Thus, the engineers or agency can make a statement such as, “Our design will avoid the flooding that resulted from the storm of August 20, 1972.”
6. BMP design may be performed with more accurate knowledge of the relationship between the magnitude of inflow rate or device volume and the percentage of annual runoff volume controlled by the BMP.

SUMMARY A wealth of analytical options exists for application to quantity problems in urban hydrology. These range from the simple calculations of the rational method (properly applied) to sophisticated computer models. However, only the latter option permits the use of continuous simulation, which avoids knotty problems of antecedent conditions and assignment of frequency to hydrographs that are inherent in the use of IDF curves and synthetic design storms. Regardless of whether an operational model is used for analysis of urban runoff, the same hydrologic tasks as in nonurban areas must be performed. These tasks include collection and analysis of rainfall data, computation of losses, conversion of rainfall excess to runoff by any of several methods, and flow routing down the drainage system. Although many of the analytical methods that are applied to nonurban areas may also be applied to computation of urban runoff, urban areas present special problems associated with high imperviousness, fast response times, and complex hydraulic phenomena in the sewer system. New methods and models are continually revealed; the readers should maintain contact with the hydrologic literature to stay abreast of new developments in a field subject to rapid change.

PROBLEMS **6.1.** Land use and population density data are given for several cities in the accompanying table. Estimate the percent imperviousness for each city using two methods: (1) imperviousness as a function of population density and (2) weighted imperviousness as a function of land use. Assume that undeveloped land is the same as “open” in Table 6–2, and “other” is 50% institutional and 50% open. (Data sources: Heaney et al., 1977; Sullivan et al., 1978.)

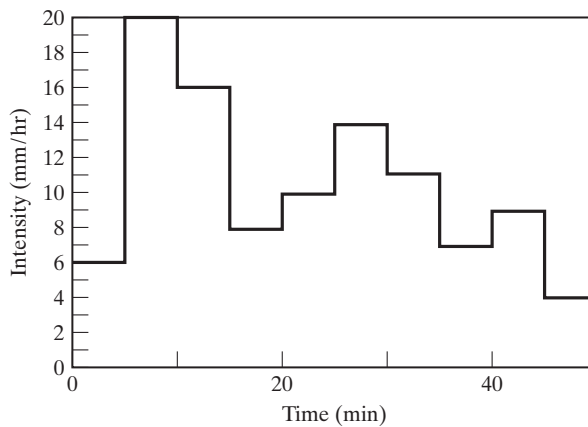
City	Urbanized Area (1000 ac)	Population Density (persons/ac)	Land Use (percent)					Total
			Residential	Commercial	Industrial	Other*	Undeveloped**	
Boston, MA	425	6.24	38.2	5.6	9.7	11.9	34.6	100
Trenton, NJ	42	6.59	39.3	5.8	10.0	12.3	32.6	100
Tallahassee, FL	19	4.06	29.1	4.3	7.4	9.1	50.1	100
Houston, TX	345	4.86	33.0	4.8	8.3	10.2	43.7	100
Chicago, IL	626	9.13	46.0	6.8	11.7	14.3	21.2	100
Denver, CO	188	5.58	35.8	5.3	9.1	11.2	38.6	100
San Francisco, CA	436	6.86	40.2	5.9	10.2	12.5	31.2	100
Windsor, Ont.	26	7.63	38.0	6.0	10.0	18.7	27.3	100

*"Other" = recreational, schools and colleges, cemeteries.

**High "undeveloped" results from the definition of urbanized area, which includes population densities as low as one person/ac.

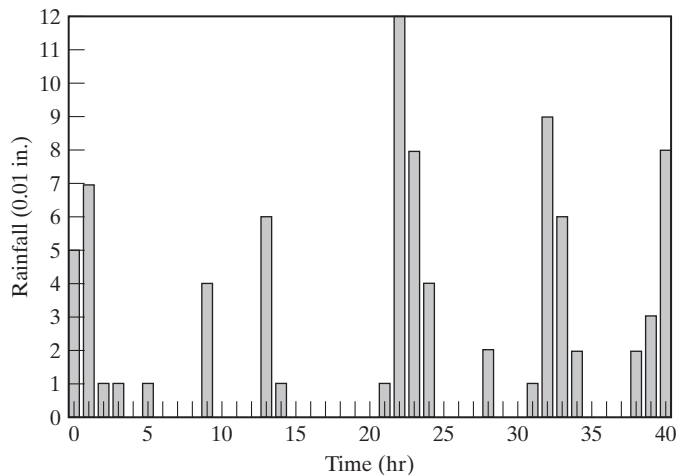
- 6.2.** A small, 2-ha, mostly impervious urban catchment has an average slope of 1.5% and the following average Horton infiltration parameters: $f_0 = 4$ mm/hr, $f_c = 1$ mm/hr, $k = 2.2$ hr⁻¹ (infiltration occurs through cracks in the paving). Consider the rainfall hyetograph in Figure P6-2.
- Determine the depression storage using Figure 6-3.
 - Determine the time of beginning of runoff.
 - Calculate and sketch the hyetograph of rainfall excess. Use the same time intervals as for the rainfall hyetograph, and average the first nonzero rainfall excess over the whole time interval.
 - Determine the runoff coefficient.
 - Compute the volume of runoff, in m³.

Figure P6-2



- 6.3.** Using the runoff volume found in Problem 6.2, determine the depth of storage in a retention basin that has vertical walls and a base area of 600 m².
- 6.4.** For the hyetograph of hourly rainfall values shown in Figure P6-4, determine the number of events corresponding to minimum interevent times (MITs) of

Figure P6-4



0, 1, 2, 3, 4, and 5 hr. What MIT is needed to have the entire 40-hr sequence treated as one event?

- 6.5. The EPA SYNOP program (Environmental Protection Agency, 1976; Hydroscience, 1979; SWMM4 Rain block—Huber and Dickinson, 1988) has been run for hourly rainfall data for Houston for the period 1948–1979. A minimum interevent time of 16 hr was used to separate independent storm events, giving the following results:

Rank	Date	Rainfall Depth (in.)	Duration (hr)
1	06/11/73	11.55	52
2	06/24/60	11.33	63
3	10/11/70	7.15	17
4	10/14/57	6.78	40
5	11/12/61	6.59	22
6	07/14/49	6.33	53
7	06/20/63	5.69	11
8	04/14/66	5.49	17
9	09/09/71	5.45	29
10	09/04/73	5.28	41
11	07/09/61	5.22	77
12	04/14/73	4.81	47
13	10/22/70	4.80	21
14	10/06/49	4.60	46
15	07/29/54	4.55	24
16	09/19/67	4.43	47
17	08/24/67	4.38	27
18	03/20/72	4.22	7
19	10/31/74	4.10	34
20	05/12/72	3.98	4
21	05/21/70	3.90	10
22	06/17/68	3.86	8
23	08/02/71	3.84	46
24	05/15/70	3.74	28
25	07/07/73	3.70	11
26	12/10/63	3.60	94
Check Sums		139.37	876

	Depth (in.)	Duration (hr)	Log ₁₀ Depth
Mean	5.37	33.4	0.7082
Unbiased standard Deviation	2.01	22.2	0.1325
Unbiased skew	2.150	0.938	1.355

- (a) Perform a log Pearson type 3 frequency analysis (Chapter 3) to determine the storm volume magnitudes corresponding to return periods of 2, 5, 10, 25, 50, and 100 yr. Use the method-of-moments fit.

- (b) Compare the volumes of part (a) with the 24-hr volumes indicated on the Houston IDF curve shown in Figure 1–15. The fair agreement between the LP3-fitted storm event depths and the 24-hr IDF depths indicates that the IDF curves were probably derived from these very same data, extracting the largest 24-hr depth out of a much longer storm event (see the durations listed in the preceding table).
 - (c) Plot storm event depths vs. storm event durations. Compute the coefficient of determination (R^2). The linear relationship between event depths and event durations is very poor and is not significant at the 95% level.
- 6.6.** Using the IDF curves for Houston shown in Figure 1–8 and the SCS hyetograph distribution given in Table E6–4, prepare a 25-yr SCS type II design storm for Houston. Plot the hyetograph of hourly values.
- 6.7.** Consider a circular pipe of diameter 1 m and a trapezoidal channel of maximum depth 1 m. The trapezoidal channel has a bottom width of 1 m and side slopes (vertical/horizontal) of 0.25.
- (a) For a Manning roughness of 0.020 and slope of 0.008 for each channel, calculate the water velocity under uniform flow at depths of 0.25, 0.50, and 0.75 m.
 - (b) Calculate the dynamic wave speeds (in downstream direction) in each channel for depths of 0.25, 0.50, and 0.75 m.
 - (c) For each wave speed and each channel, calculate the travel time over a length of 300 m.
- 6.8.** A planned 5.68-ac subdivision is sketched in Figure P6–8. The soils are generally sandy, and the only runoff will occur from the directly connected (i.e., hydraulically effective) impervious street and driveway surfaces shown in the figure. (Only the 20×30 ft portion of each driveway that drains to the street is shown.) The street is 30 ft wide and the cul-de-sac has a radius of 30 ft. For storm drainage, the plan is to let the stormwater run along the street gutters in lieu of installing a pipe. For purposes of this problem, the entire drainage system can be treated as overland flow. The street slopes from an elevation of 165 ft at the center of the cul-de-sac to 160 ft at the entrance to the subdivision and has a Manning roughness of 0.016.

Drainage regulations specify a 5-yr return period design. Local IDF curves can be approximated functionally as

$$i = \frac{a}{b + t_r},$$

where

i = rainfall intensity (in./hr),

t_r = duration (min),

a, b = constants for different return periods.

For this hypothetical location, $a = 160$ and $b = 18$ for a return period of 5 yr.

- (a) Estimate the directly connected impervious area (ac).
- (b) Estimate the maximum drainage length along the DCIA (ft).

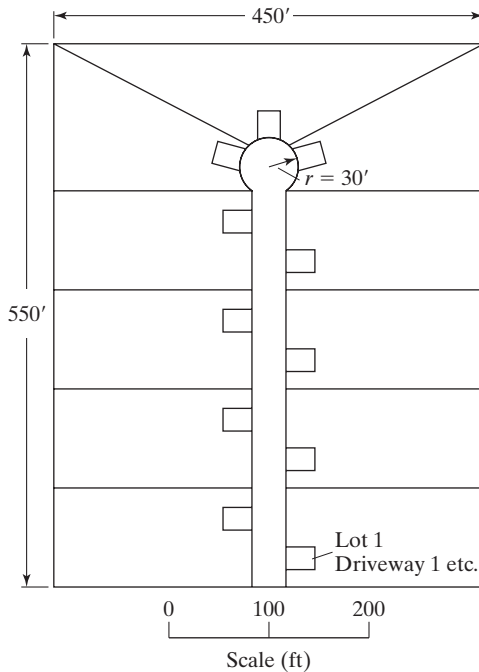


Figure P6-8

- (c) Determine the kinematic wave parameters α and m .
- (d) Estimate the 5-yr peak flow at the outlet. Assume that the impervious surface experiences no losses and ignore the geometry of street gutters (treat the street as a flat surface).
- 6.9.** The subdivision of Problem 6.8 drains to the upstream end of a circular pipe that has an n value of 0.013 and a slope of 0.006 ft/ft. Determine the size of pipe needed to carry the flow from the 5-yr storm. (*Note:* Standard diameters in the United States start at 12 in. and increase in 6-in. increments to 60 in., then continue to increase in 12-in. increments.)
- 6.10.** A 14.7-ac multifamily residential catchment in Miami has a total impervious area of 10.4 ac, but it has a hydraulically effective impervious area of only 6.48 ac. The pervious portions of the basin consist of lawns over a Perrine marl, with a very slow infiltration rate. Rainfall and runoff data monitored by the USGS (Hardee et al., 1979) are reported in Table 6-9 for 16 storm events.
- (a) Determine a linear relationship between runoff and rainfall using linear regression analysis (least squares). Test the significance of the regression, and plot the data points and the fitted line.
- (b) What is the value of depression storage for this basin?
- 6.11.** Using the data of Problem 6.10, determine the average runoff coefficient by
- (a) computing the runoff coefficient for each storm and finding the average;
- (b) dividing the total runoff for all storms by the total rainfall for all storms;

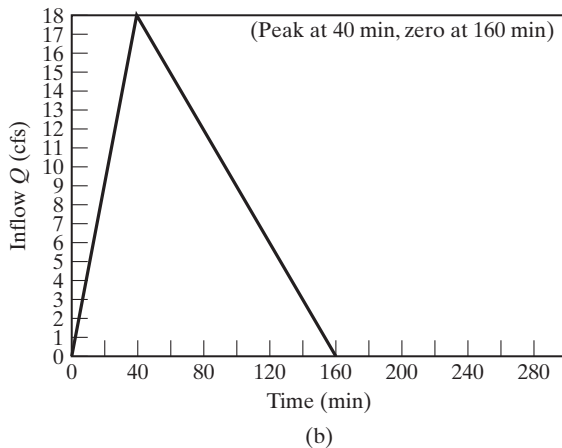
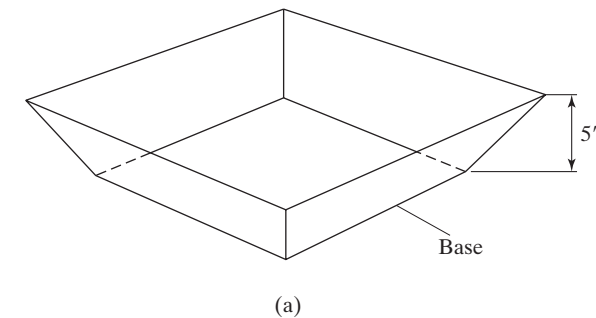
- (c) finding the slope of a runoff-vs.-rainfall regression line that is “forced” through the origin.

Discuss the computed runoff coefficients in relation to the values of imperviousness and hydraulically effective imperviousness.

- 6.12.** A detention pond has the shape of an inverted truncated pyramid, shown in Figure P6–12(a). It has a rectangular bottom of dimension 120×80 ft, a maximum depth of 5 ft, and uniform side slopes of 3:1 (horizontal:vertical). Hence the dimensions at 5-ft depth are also rectangular, with length 150 ft and width 110 ft. The outlet from the basin behaves as an orifice, with a diameter of 1 ft and a discharge coefficient of 0.9. The opening of the orifice (a pipe draining from the center of the basin) is effectively at a depth of zero (i.e., at the bottom of the pond). (The pond floor would typically slope toward the outlet, but this slope will be ignored in this problem. In addition, the orifice will be assumed to follow its theoretical behavior even at very small depths.)

- (a) What is the total volume of the pond, in ft^3 and ac-ft?
- (b) Develop the depth-vs.-surface-area and depth-vs.-volume curves for the pond. Use 1-ft intervals. Tabulate and plot.

Figure P6–12



(c) A triangular inflow hydrograph is shown in Figure P6-12(b). Route it through the detention pond. The storage indication method (Puls method) of Section 4.3 is recommended, with a time step of 10 min. Plot the outflow hydrograph on the same graph as the inflow hydrograph.

6.13. Use the USGS regression equation [Eq. (3-82)] to compute the 5-yr peak flow for a 4-ac Tallahassee catchment that is 95% impervious. Compare your answer with the value computed in Example 6-6. The reason for the large discrepancy is that the regression equations were developed for much larger catchments (the smallest being 0.21 mi^2) and probably do not apply to a 4-ac catchment.

6.14. The time–area–concentration curve for an urban basin is given below. Also provided is a table giving the rainfall hyetograph and losses.

Time (min)	Incremental Area (ac)
0–10	4
10–20	20
20–30	15
30–40	8
40–50	5

Time (min)	Intensity (in./hr)	Losses (in./hr)
0–10	0.10	0.08
10–20	0.30	0.06
20–30	0.25	0.04
30–40	0.15	0.04

- What is the total area of the catchment?
- What is the time of concentration of the overall catchment?
- Compute and plot the hyetograph of rainfall excess as a bar chart.
- Plot the time–area–concentration curve as a bar chart.
- Use the time–area method to compute the runoff hydrograph, without any additional attenuation.
- Provide additional attenuation by routing the runoff hydrograph through a linear reservoir with time constant $K = 20 \text{ min}$. Use the Muskingum method (Chapter 4) to perform the routing, with routing parameter $x = 0$. When a time–area hydrograph is routed through a linear reservoir, this is known as a Clark model (1945).
- Repeat part (f) for a time constant $K = 7.5 \text{ min}$. Plot the inflow hydrograph from part (e) and the attenuated hydrographs from parts (f) and (g) on the same chart. Note the difference in attenuation for the two linear reservoirs.

- 6.15.** A catchment is to be simulated using the Clark model—that is, by routing using a time–area method (to produce hydrograph time delays), followed by routing through a linear reservoir (to produce hydrograph attenuation). The time–area and rainfall–excess data are given below:

Time (min)	Area (ac)	Time (min)	Rainfall Excess (in./hr)
0–30	8	0–30	0.35
30–60	42	30–60	0.25
60–90	40	60–90	0.30
90–120	11	90–120	0.10
120–150	19		

- What is the total area of the catchment?
 - What is the time of concentration of the overall catchment?
 - Perform the indicated time–area routing.
- 6.16.** Compare and contrast the features of SWMM model and HEC-HMS (see Chapter 5).
- 6.17.** Monitoring data for an Oregon catchment produce the following record of annual precipitation and runoff:

Precipitation, in./yr	Runoff, 1000-ac-ft
33	650
42	940
50	1010
35	740
36	620
44	850
43	980
48	910
37	800
42	990
49	1280
40	880
41	1070
55	1330
45	970
39	800
46	1040
Check Sums: 725	15,860

- (a) Determine a linear relationship between runoff and precipitation using linear regression analysis (least squares). Test the significance of the regression, and plot the data points and the fitted line.
- (b) Does this catchment exhibit the characteristics of depression storage or of baseflow?
- (c) Do you think this is a large catchment or a smaller one? Assume “large” is an area greater than 100 square miles. Base your assessment on an evaluation of the slope of the fitted line and its units.

6.18. A 9.6-acre low-density residential site in Anchorage, Alaska, was monitored by the USGS during a study of the hydrology and water quality characteristics of the area (Brabets, 1987). Data for rainfall depth, runoff depth, and suspended solids, SS (in units of lb and lb/ac-in.) are given in the table. These data are for discrete rainfall events. Note that pounds per acre-inch has units of mass/volume and is a concentration, whereas pounds of SS are a load (mass) and calculated by multiplying concentration $\times$ volume of runoff. (Appropriate conversion factors are included, of course.)

Rain (in.)	Runoff (in.)	SS (lb)	SS (lb/ac-in.)
0.08	0.024	12.4	53.8
0.08	0.010	6.4	66.7
0.31	0.060	6.0	10.4
0.50	0.133	9.6	7.5
0.19	0.030	3.9	13.5
0.23	0.110	5.1	4.8
0.30	0.090	8.3	9.6
0.22	0.060	5.7	9.9
0.11	0.025	21.1	87.9
0.13	0.029	19.0	68.2
0.16	0.026	9.7	38.9
0.18	0.046	17.8	40.3
0.17	0.041	4.3	10.9
0.48	0.129	57.4	46.4
0.10	0.039	22.5	60.1
0.06	0.010	1.2	12.5
0.14	0.022	5.0	23.7
0.54	0.133	55.4	43.4
0.08	0.037	9.5	26.7
0.55	0.090	17.1	19.8
0.23	0.059	22.9	40.4
0.23	0.042	16.9	41.9
0.16	0.040	6.5	16.9
0.35	0.110	25.6	24.2
Check Sums: 5.58	1.395	369.3	778.4

- (a) Demonstrate the units conversion computation for the first row in the table (i.e., that 53.8 lb/ac-in. with a runoff of 0.024 in. results in 12.4 lb of SS).

- (b) Rainfall is to be considered as an explanatory variable (independent variable) for the prediction of runoff, SS (lb), and SS (lb/ac-in.). Perform the three indicated linear regressions. Test the significance of the regressions at the 95% level ($\alpha = 5\%$). Plot the data points for runoff vs. rainfall on one graph and for both SS values vs. rainfall on one or two other graphs. If the regression is significant, include the predicted straight line.

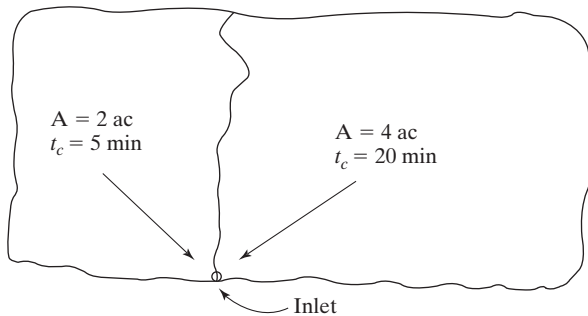
Although your software may test the significance automatically, list the “table” T -value that must be exceeded for the regression to be significant. Obtain this value from a statistics book.

- (c) What other causative factors (that would vary with each storm) might be included in a multiple linear regression of runoff vs. rainfall (depths)?

Note: This problem illustrates an example of “spurious correlation” for the SS-vs.-rainfall data. Load is the product of a constant $\times$ runoff depth $\times$ concentration. Since runoff is correlated with rainfall, the dependent variable (load) “includes” rainfall as part of its value. Hence, load will always correlate better with rainfall than will concentration.

- 6.19.** A 6-ac basin is to be developed into 2 ac of commercial development ($C = 0.95$) and 4 ac of park ($C = 0.2$), as sketched in Figure P6–19. Using the tabulated IDF information, what should be the design flow at the inlet?

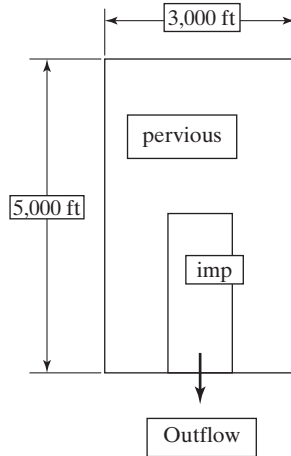
Figure P6–19
Sketch of catchment
for Problem 6.19.



Duration (min)	Intensity (in./hr)
5	11.1
10	8.2
15	7.5
20	8.6
25	6.5
30	5.8

- 6.20.** A planned subdivision near Tallahassee can be represented conceptually as shown in Figure P6–20.

The inner rectangle represents DCIA, consisting of rooftops, streets, sidewalks, driveways, and so on. The dimensions of the inner rectangle are 1000 ft wide by 2500 ft tall. The pervious area behaves like “lawns, sandy soil, average

**Figure P6–20**

Conceptual sketch of catchment, with DCIA “lumped” at outlet.

slope.” The DCIA has characteristics of streets and roofs. “High-end” values of runoff coefficients should be used.

The inlet time for flow from the far upper end of the catchment (e.g., an upper corner of the pervious area) is 2 hr. The inlet time just for the DCIA is 20 min.

- What is the 25-yr outflow, by the rational method? Remember to compute runoff both for the total area and for just the DCIA.
- Use the USGS regression equations for Tallahassee [Eq. (3–82)] and compute the 25-yr peak flow. Observe that this value is close to the rational method estimate for DCIA, from part (a).
- A corrugated metal pipe (CMP) culvert ($n = 0.024$) will receive the 25-yr flow from the catchment, using the rational method of part (a). If the pipe slope is 1%, what standard U.S. diameter (see Problem 6.9) should be used?

6.21. A subdivision in Corvallis, Oregon, is shown in Figure P6–21. Rational method design is performed using the 10-yr return period IDF curve of the Oregon Department of Transportation (2005) for Zone 8, which may be approximated as follows:

$$i = 0.2081 + \frac{29.8438}{t_r} - \frac{184.51}{t_r^2} + \frac{432.8875}{t_r^3},$$

with i in in./hr and t_r in minutes. The standard error of estimate is 0.0079 in./hr.

- Estimate the peak flow at the outlet from the “curved end portion” of Sitka Place, roughly where a line on the asphalt pavement would connect houses 3 and 8, as shown by the arrow on the figure. Follow these steps and guidelines:
 - You must construct an estimate of the tributary area.
 - The impervious areas of houses 4, 5, 6, and 7 contribute runoff to flow at the upper end of the street, but none of the other houses do. Thus,

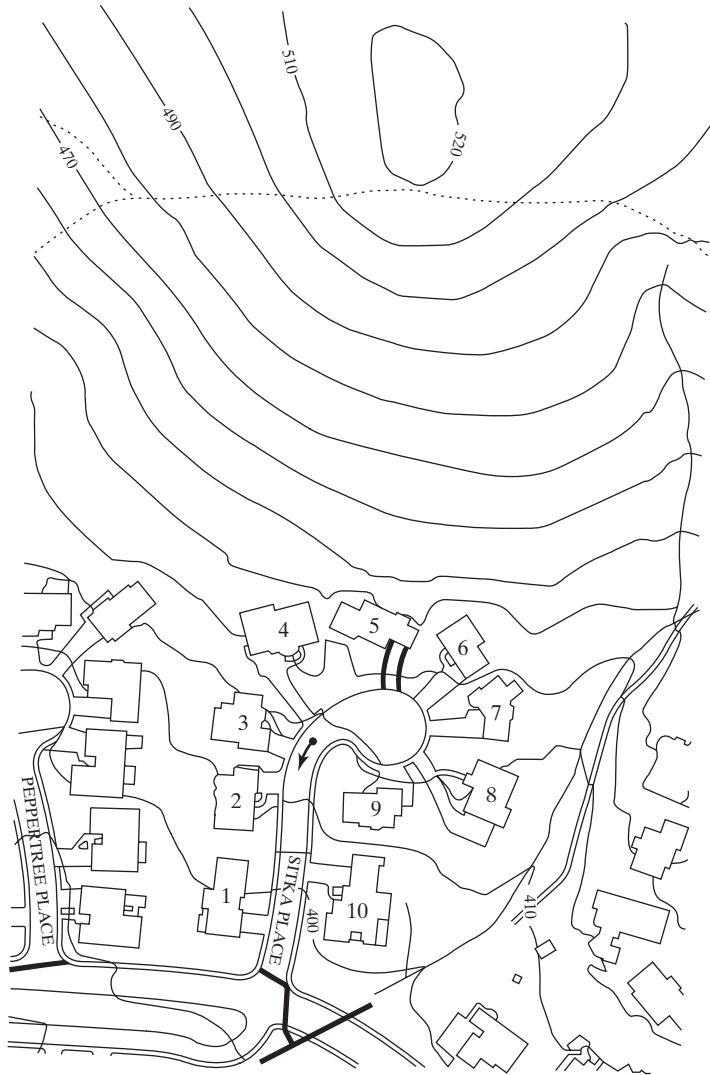


Figure P6-21

Subdivision for Problem 6.21, with 10-ft contours. All impervious area is directly connected to the drainage system. To set a scale, the distance between the upper left corner of house 4 and lower right corner of house 10 is 381 ft. The outside dimensions of the whole figure are approximately 708 ft by 1104 ft.

you can assume that the downhill edges of houses 4 and 7 and the downstream edge of their driveways define the lower boundary of the basin (apart from the pavement of the cul-de-sac).

- Work uphill from these lower boundaries toward the high point on the hill to define the tributary area, keeping in mind that the boundary

should be perpendicular to the contours. This will define a “teardrop”-shaped catchment.

- Estimate the area using a planimeter or by counting squares. Estimate an average slope by dividing the difference in upper and lower elevations by an estimated path length.
 - The undeveloped area above the end of the street behaves like a lawn on “heavy soil.” Overland flow roughness for this surface could be characterized as for “dense shrubbery and forest litter.”
 - Use the kinematic wave method to estimate t_c and hence the peak flow. Assume that the travel time over the impervious area is much less than the travel time over the pervious area. Thus, base kinematic wave parameter α only on the pervious area characteristics. Do not compute a weighted Manning’s roughness.
 - The limit for the intensity-vs.-duration equation is $3 \leq t_r \leq 400$ minutes. Stop computation at 3 minutes if indicated.
- (b) All the impervious area (rooftops, driveways, streets) is directly connected. Hence, compute a second estimate of peak flow based only on the DCIA. Which estimate governs?
- (c) The AutoCAD map of Figure P6–21 was prepared in 1991. But in the year 2007, the hill above Sitka Place was completely developed with new houses! What does this imply about hydrologic design for areas such as this?

6.22. The Santa Barbara Urban Hydrograph Method (SBUHM) consists of two basic computational steps:

- (1) Compute rainfall excesses for each rainfall time step. Rainfall excesses (depth/time) from the pervious and impervious areas are multiplied by their respective areas and summed to obtain one flow value per time step for the entire catchment (the “instantaneous hydrograph”). Any convenient means may be used to compute rainfall excess, ranging from very simple runoff coefficients with depression storage [Eq. (6–11)], to SCS curve numbers (Chapter 2), to the infiltration equations of Chapter 2. In this method, DCIA is often assumed to have no losses.
- (2) The resulting instantaneous hydrograph is then routed through an imaginary linear reservoir to provide delay and attenuation. The linear reservoir has a time constant $K = t_c$ for the catchment. Routing is performed using the Muskingum method equations (Section 4.2), with Muskingum parameter $x = 0$ (for a linear reservoir) and $\Delta t =$ time step of instantaneous hydrograph.

Use the data from Problem 6.15 to perform SBUHM routing. Assume that the rainfall excess applies to the entire catchment area (so for this problem, there is no merging of runoff from pervious area and DCIA).

- (a) Compute the instantaneous hydrograph at 30-min intervals, in units of cfs $\approx$ ac-in./hr.

- (b) Inferring a value for the time of concentration for the overall catchment from the given time–area data, compute Muskingum coefficients C_0 , C_1 , and C_2 , with Muskingum $K = t_c$ and $x = 0$ for the linear reservoir.
- (c) Perform the indicated flow routing through the hypothetical reservoir. Plot the inflow “instantaneous hydrograph” and outflow hydrograph on the same graph. Remember that the hydrographs begin with zero flow.
- (d) If you have worked Problem 6.15, compare the hydrographs by the two methods.

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Chapter 7

Floodplain Hydraulics



White Water Rafting in Browns Canyon, Colorado.



Hood River near junction with Columbia River, Oregon.

This chapter on **open-channel flow** is limited to coverage of steady-flow problems, such as **uniform flow** and **nonuniform flow**. Unsteady channel flow hydraulics is beyond the scope of this text but is available in more advanced fluid mechanics references. Channel and floodplain computations of **water surface profiles** are critical to defining levels of flood inundation and are normally treated as steady-flow problems.

Uniform open-channel flow is the hydraulic condition in which the water depth and the channel cross section do not change over some reach of the channel. These criteria require that the energy grade line, water surface, and channel bottom are all parallel. In other words, the total energy change over the channel reach is exactly equal to the energy losses of boundary friction and turbulence.

Uniform flow is eventually established over any long channel reach that has constant flow and an unchanging channel cross section. Strict uniform flow is rare in natural streams because of the constantly changing channel conditions. But it is often assumed in natural streams for engineering calculations, with the understanding that the results are general approximations of the actual hydraulic conditions. Uniform flow assumptions should not be made in cases where a well-defined and consistent channel does not exist and where rapidly varying flow is caused by changes in the channel. Man-made channels are usually very consistent, and uniform flow calculations are much more exact for them.

7.1 UNIFORM FLOW

Two uniform flow equations are frequently used for open-channel flow problems: the Chezy formula and Manning's equation, which relate the following variables:

A = cross-sectional area of channel,

V = channel velocity,

P = the wetted perimeter of the channel,

R = hydraulic radius, or cross-sectional area A divided by the wetted perimeter P ,

S = energy slope, which is equal to the slope of the channel bed under the uniform flow assumptions,

C or n = roughness coefficient, related to the friction loss associated with water flowing over the bottom and sides of the channel.

The Chezy formula was developed in 1775; it relates channel velocity to roughness, hydraulic radius, and channel slope:

$$V = C\sqrt{RS}. \quad (7-1)$$

The C factor can be related to Darcy's friction factor f , used in pipe flow, with the relationship

$$C = \sqrt{8g/f}, \quad (7-2)$$

where g is the gravitational constant.

The Chezy equation is based on two main assumptions: the frictional force is proportional to the square of the velocity, and the uniform flow assumption that the gravity force is balanced by the frictional resistance of the stream. The Chezy equation has been used for pressure pipe flow as well as open channel flow. The Chezy C coefficient for open channel conditions can be calculated using relationships from Chow (1959).

For most present applications, however, Manning's equation is used instead of the Chezy formula for open-channel computations. Manning's equation was presented in 1890; it describes roughness with the Manning's roughness coefficient n :

$$V = \frac{1}{n} R^{2/3} \sqrt{S}. \quad (7-3)$$

The roughness coefficients were originally developed in the metric system (m and s), and a conversion coefficient must be used to apply Manning's equation with U.S. customary units:

$$V = \frac{1.49}{n} R^{2/3} \sqrt{S}. \quad (7-4)$$

The conversion coefficient 1.49 is the cube root of 3.28, which converts m to ft. A dimensional analysis of n shows units of $TL^{-1/3}$ and illustrates a theoretical problem with Manning's equation. The empirical nature of the relationship does not restrict its usefulness, however, and it is widely used to solve numerous types of open-channel flow problems.

The selection of the roughness coefficient n is usually based on "best engineering judgment" or on values prescribed by municipal design ordinances. Several tables are available in the general literature for the selection of Manning's roughness coefficient for a particular open channel (see Table 7-1 or Chow, 1959).

Table 7-1. Values of the Channel Roughness Coefficient n

Type of Channel and Description	Minimum	Normal	Maximum
A. Closed conduits flowing partly full			
A-1. Metal			
a. Steel			
1. Riveted and Spiral	0.013	0.016	0.017
b. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
c. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Cement			
1. Mortar	0.011	0.013	0.015
b. Concrete			
1. Culvert, straight, and free of debris	0.010	0.011	0.013
2. Culvert, with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlets, and so on, straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
c. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
d. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, and so on	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
e. Brick work			
1. Glazed	0.011	0.013	0.015
f. Sanitary sewers coated with sewer slimes, with bends and connections	0.012	0.013	0.016
g. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
h. Rubble masonry, cemented	0.018	0.025	0.030

Table 7-1. (Continued)

Type of Channel and Description	Minimum	Normal	Maximum
B. Lined or built-up channels			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Wood			
1. Planed, untreated	0.010	0.012	0.014
b. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Gunite, wavy section	0.018	0.022	0.025
c. Concrete bottom float finish with sides of			
1. Random stone in mortar	0.017	0.020	0.024
2. Cement rubble masonry	0.020	0.025	0.030
3. Dry rubble or riprap	0.020	0.030	0.035
d. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
C. Excavated or dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
d. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Dense brush, high stage	0.080	0.100	0.140
D. Natural streams			
D-1. Minor streams (top width at flood stage < 100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more effective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060

(Continued)

Table 7-1. Values of the Channel Roughness Coefficient n

Type of Channel and Description	Minimum	Normal	Maximum
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools or floodways with heavy loads of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged in high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
D-2. Flood plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.100
D-3. Major streams (top width at flood stage > 100 ft. The n -value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	—	0.060
b. Irregular and rough section	0.035	—	0.100

Uniform flow problems usually involve the application of Manning's equation to compute the **normal depth** y_n , the only water depth at which flow is uniform. The selection of Manning's n requires more judgment and experience on the part of the engineer or hydrologist than any other parameters in the equation. One usually solves for normal depth as a function of given bed slope S_0 , flow rate, and channel geometry. Likewise, one can solve for the design width of a channel when the normal depth is specified. Various types of nomographs, tables, and computer programs are available for the solution of open-channel flow problems for channels of varying shape and dimension.

7.2 UNIFORM FLOW COMPUTATIONS

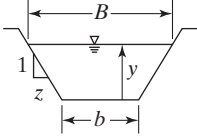
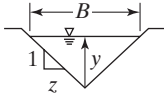
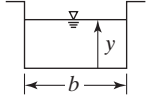
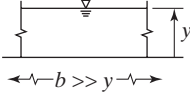
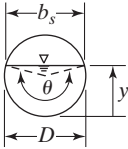
Shape	Section	Flow Area A	Wetted Perimeter P	Hydraulic Radius R
Trapezoidal		$y(b + zy)$	$b + 2y\sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}}$
Triangular		zy^2	$2y\sqrt{1 + z^2}$	$\frac{zy}{2y\sqrt{1 + z^2}}$
Rectangular		by	$b + 2y$	$\frac{by}{b + 2y}$
Wide flat		by	b	y
Circular		$(\theta - \sin \theta) \frac{D^2}{8}$	$\frac{\theta D}{2}$	$\frac{D}{4} \left(1 - \frac{\sin \theta}{\theta}\right)$

Figure 7-1

Geometric properties of common open-channel shapes.

A cross section can be characterized by its shape, normal depth, cross-sectional area, and hydraulic radius, defined as the ratio of area to wetted perimeter. Figure 7-1 indicates the properties of various geometries of open channel flow sections. Depending on the shape of the sections, Manning's equation can be used to solve for normal depth or width, given the other parameters. Examples 7-1 and 7-2 indicate the solution of uniform flow problems for a rectangular and trapezoidal open channel, respectively.

EXAMPLE 7-1**UNIFORM FLOW IN A RECTANGULAR CHANNEL**

A rectangular open channel is to be designed to carry a flow of $2.28 \text{ m}^3/\text{s}$. The channel is to be built using concrete (Manning's $n = 0.014$) on a slope of 0.006 m/m . Assuming normal flow in the channel, determine the normal depth y_n if $b = 2.0 \text{ m}$ (see Fig. E7-1).

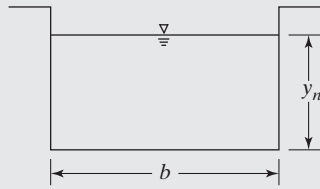


Figure E7-1

Given

$$Q = 2.28 \text{ m}^3/\text{s},$$

$$n = 0.014,$$

$$S_0 = 0.006 \text{ m/m},$$

$$b = 2 \text{ m},$$

Equation (7-3) gives

$$V = \frac{1}{n} R^{2/3} \sqrt{S_0}$$

for metric units. It is known that $Q = VA$, so

$$Q = \frac{1}{n} AR^{2/3} \sqrt{S_0}.$$

Knowing that

$$R = A/P$$

and referring to the figure, we have

$$A = y_n b = 2.0 y_n,$$

$$P = 2y_n + 2.0.$$

Therefore,

$$Q = \frac{1}{n} AR^{2/3} \sqrt{S_0},$$

$$2.28 = \frac{1}{0.014} (2y_n) \left(\frac{2y_n}{2y_n + 2} \right)^{2/3} \sqrt{0.006},$$

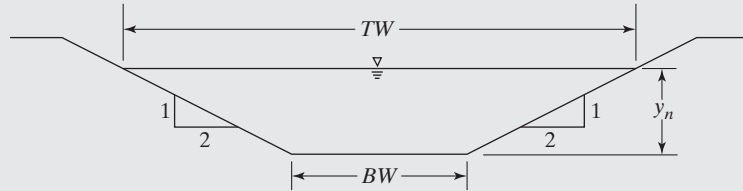
$$0.413 = \left(\frac{1}{2y_n + 2} \right)^{2/3} (2y_n)^{5/3}$$

$$y_n = 0.45 \text{ m}.$$

SOLUTION

EXAMPLE 7-2**UNIFORM FLOW IN A TRAPEZOIDAL CHANNEL**

A trapezoidal channel with side slopes of 2 to 1 is designed to carry a normal flow of 200 cfs. The channel is grass-lined with a Manning's n of 0.025 and has a bottom slope of 0.0006 ft/ft. Determine the normal depth, bottom width, and top width (see Fig. E7-2) assuming normal flow and the bottom width (BW) as 1.5 times the normal depth.

Figure E7-2**SOLUTION**

Given

$$Q = 200 \text{ cfs,}$$

$$n = 0.025,$$

$$S_0 = 0.0006 \text{ ft/ft,}$$

$$BW = 1.5y_n,$$

Equation (7-4) gives

$$V = \frac{1.49}{n} R^{2/3} \sqrt{S_0}.$$

Referring to the figure, we have

$$\begin{aligned} P &= BW + 2(y_n^2 + 4y_n^2)^{1/2} \\ &= 1.5y_n + 2(\sqrt{5}y_n) \\ &= y_n(1.5 + 2\sqrt{5}) \end{aligned}$$

and

$$\begin{aligned} A &= BWy_n + 2(1/2)(2y_n)(y_n) \\ &= 1.5y_n^2 + 2y_n^2 \\ &= 3.5y_n^2. \end{aligned}$$

Then

$$\begin{aligned} R &= A/P \\ &= \frac{(3.5y_n^2)}{(y_n)(1.5 + 2\sqrt{5})} \\ &= 0.586y_n, \end{aligned}$$

so

$$Q = \frac{1.49}{n} AR^{2/3} \sqrt{S_0},$$

$$200 = \frac{1.49}{0.025} (3.5y_n^2) (0.586y_n)^{2/3} \sqrt{0.0006},$$

$$200 = 3.578y_n^{8/3},$$

$$y_n = (55.89)^{3/8},$$

$$y_n = 4.5 \text{ ft.}$$

Then,

$$BW = 6.8 \text{ ft,}$$

$$TW = 24.8 \text{ ft.}$$

For a given slope, flow rate, and roughness, an **optimum channel cross section** can be found that requires a minimum flow area. The optimum cross section is the one for which the hydraulic radius R is a maximum and the wetted perimeter is a minimum, since $R = A/P$. Figure 7-2 indicates the properties of optimum open-channel sections based on minimizing the wetted perimeter for each shape.

Shape	Section	Optimum Geometry	Normal Depth y_n	Cross-Sectional Area A
Trapezoidal (half hexagon)		$\theta = 60^\circ$ $b = \frac{2}{\sqrt{3}} y_n$	$0.968 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/8}$	$1.622 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/4}$
Rectangular (half square)		$b = 2y_n$	$0.917 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/8}$	$1.682 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/4}$
Triangular (half square)		$\theta = 45^\circ$	$1.297 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/8}$	$1.682 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/4}$
Circular (half full)		$D = 2y_n$	$1.00 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/8}$	$1.583 \left[\frac{Qn}{S_b^{1/2}} \right]^{3/4}$

Figure 7-2

Properties of optimum open-channel sections.

EXAMPLE 7-3**UNIFORM FLOW IN AN URBAN STREAM**

Brays Bayou can be represented as a single trapezoidal channel with a bottom width b of 75 ft and a side slope of 4:1 (horizontal:vertical) on average. If the normal bankfull depth is 25 ft at the Main Street bridge, compute the normal flow rate in cfs for this section. Assume that $n = 0.020$ and $S_0 = 2.0 \times 10^{-4}$ for the concrete-lined channel.

Given

$$y_n = 25 \text{ ft},$$

$$n = 0.02,$$

$$S_0 = 0.0002,$$

$$b = 75 \text{ ft},$$

Equation (7-4) is used to compute Q , and from the geometry provided,

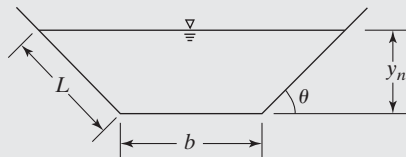
$$\begin{aligned} Q &= \frac{1.49}{n} AR^{2/3} \sqrt{S_0} \\ &= \left(\frac{1.49}{0.02} \right) \frac{A^{5/3}}{P^{2/3}} \sqrt{0.0002}, \end{aligned}$$

$$A = 4375 \text{ ft}^2 \text{ and } P = 281.16 \text{ ft}.$$

Thus,

$$Q = 28,730 \text{ cfs}.$$

Figure E7-3



**7.3
SPECIFIC ENERGY
AND CRITICAL
FLOW**

Specific energy is a case of **total energy** that can be defined for any location along an open channel. The total energy can be defined as the sum of the pressure head, the elevation head, and the velocity head for any cross section. In its simplest form, the energy equation becomes

$$H = \frac{p}{\gamma} + z + \frac{V^2}{2g}, \quad (7-5)$$

where $\gamma = \rho g$ and $y = p/\gamma = \text{depth}$. Specific energy E at a single section is referenced to the channel bed and therefore is a sum of depth y and **velocity head** $V^2/2g$:

$$E = y + \frac{V^2}{2g}, \quad (7-6)$$

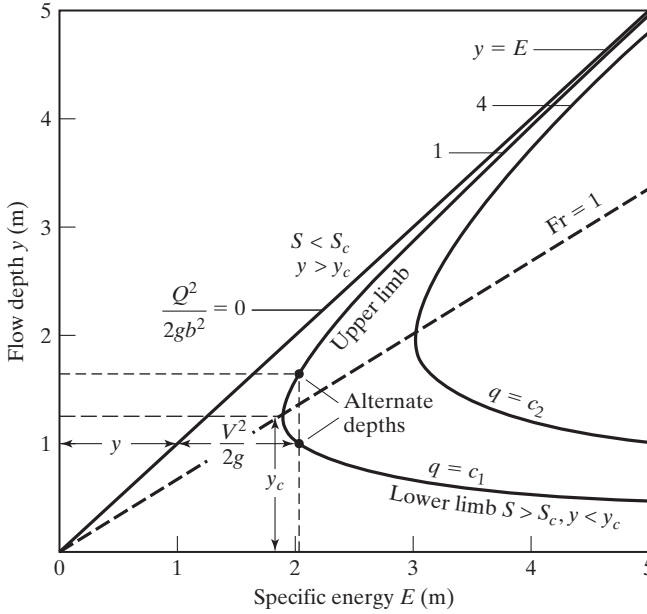


Figure 7-3
Specific energy
diagram.

where the depth y is measured normal to the channel bed. For uniform flow at a section, specific energy can be written in terms of flow rate Q by substituting $V = Q/A$ in Equation (7-6):

$$E = y + \frac{Q^2}{2gA^2}. \quad (7-7)$$

For simplicity, the following discussion is limited to a wide rectangular channel where $V = Q/A = q/y$, where q is flow per unit width in the open channel Q/b . Thus E can be written as a function of y only:

$$E = y + \frac{Q^2}{2gb^2y^2}, \quad (7-8)$$

where b is channel width.

Figure 7-3 shows the variation of depth as a function of E for a given flow rate. It can be seen that for a given flow rate and specific energy, there are two possible values of depth y , called alternate depths. The curve for constant q gives a curve of depth values and the corresponding values of E . As q increases, the curves are shifted to the right. For each curve in Figure 7-3, there is a value of depth y_c that gives a minimum E , which may be determined by differentiating Equation (7-8) and setting the result to 0:

$$\frac{dE}{dy} = 1 - \left(\frac{q^2}{gy^3} \right). \quad (7-9)$$

Solving for y , we obtain the **critical depth** y_c , defined as

$$y_c = \left(\frac{q^2}{g} \right)^{1/3}. \quad (7-10)$$

Summarizing the results for a rectangular channel, **critical flow** can be characterized by the following relationships:

$$E_{\min} = \left(\frac{3}{2}\right)y_c = \left(\frac{V_c^2}{2g}\right) + y_c,$$

$$\frac{V_c^2}{2g} = \frac{y_c}{2}$$

Thus,

$$\text{Fr} = \frac{V_c}{\sqrt{gy_c}} = 1, \quad (7-11)$$

$$y_c = \left(\frac{q^2}{g}\right)^{1/3}.$$

The critical condition occurs when **Froude number** (Fr) is equal to 1, where $\text{Fr} = V/\sqrt{gy}$. The two arms of the curve in Figure 7-3 provide additional information for classifying open-channel flows. On the upper limb of the curve, the flow is said to be tranquil, while on the lower limb it is called rapid flow. The velocity and flow rate occurring at the critical depth are termed V_c and q_c , the critical velocity and critical flow, respectively. The velocity of the upper limb flow is slower than critical and is called **subcritical** velocity, and the velocity of the lower limb flow is faster than critical and is called **supercritical** velocity.

The Froude number is used to characterize open-channel flow, and $\text{Fr} < 1$ occurs for subcritical flow and $\text{Fr} > 1$ for supercritical flow. From Equation (7-11) a condition of subcritical or supercritical flow may be easily tested by comparing the velocity head $V^2/2g$ to the value of $y/2$. For any value of E there exists a critical depth for which the flow is a maximum. For any value of q there exists a critical depth for which specific energy is a minimum. For any flow condition other than critical, there exists an alternate depth at which the same rate of discharge is carried by the same specific energy. Alternate depth can be found by solving Equation (7-8).

The condition of critical flow just defined is used to define channel slope. If, for a given roughness and shape, the channel slope is such that the uniform flow is subcritical, slope is said to be **mild** and $y > y_c$. If the uniform flow is supercritical, the slope is termed **steep** and $y < y_c$. A **critical slope** S_c is the slope that will just sustain a given rate of discharge in a uniform flow at critical depth. When flow is near critical, a small change in E results in a large change in depth, and an undulating stream surface will result. Because of this phenomenon, shown in Figure 7-3, it is undesirable to design channels with slopes near the critical condition.

For nonrectangular channels, the specific energy equation becomes

$$E = y + \frac{Q^2}{2gA^2}, \quad (7-12)$$

where area $A = f(y)$. Differentiating with respect to y and realizing that $dA = B dy$, where B is the top width of the water surface, we obtain

$$\frac{dE}{dy} = 1 - \frac{Q^2}{2g} \left(\frac{2}{A^3} \frac{dA}{dy} \right)$$

or

$$\frac{Q^2}{g} = \left(\frac{A^3}{B} \right)_{y=y_c} \quad (7-13)$$

The Froude number for a nonrectangular channel is defined as $Fr = V/\sqrt{gD}$, where D is the hydraulic depth, or A/B . If the channel is rectangular, $A = By$, and the above reduces to Equation (7-10). Example 7-4 indicates the computation of critical flow conditions for an open channel based on the above equations.

CRITICAL FLOW COMPUTATION

EXAMPLE 7-4

Water is flowing uniformly in an open channel (triangular shape) at a rate of $14 \text{ m}^3/\text{s}$. The channel has 1:1 side slopes and a roughness coefficient of $n = 0.012$ (see Fig. E7-4). Determine whether the flow is subcritical or supercritical if the bottom slope is 0.006 m/m .

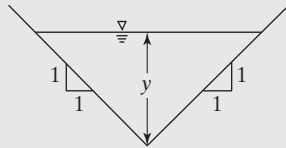


Figure E7-4

Critical depth is found using Equation (7-13), which states

SOLUTION

$$Q^2/g = (A^3/B),$$

when $y = y_c$. Referring to the figure, where B is top width, it can be seen that

$$A = y^2,$$

$$P = 2\sqrt{2}y,$$

$$R = y/(2\sqrt{2}),$$

$$B = 2y.$$

Thus for $y = y_c$,

$$A = y_c^2 \quad \text{and} \quad B = 2y_c.$$

Therefore,

$$A^3/B = Q^2/g,$$

$$y_c^6/2y_c = (14)^2/9.81,$$

$$y_c^5 = 39.96 \text{ m}^5,$$

$$y_c = 2.09 \text{ m}.$$

Since the flow is assumed to be uniform, the depth can be found using Manning's equation [Eq. (7-3)]:

$$\begin{aligned} Q &= (1/n)AR^{2/3}\sqrt{S_0}, \\ 14 &= (1/0.012)(y^2)(y/2\sqrt{2})^{2/3}(\sqrt{0.006}), \\ y^{8/3} &= 4.338 \\ y &= 1.73 \text{ m}. \end{aligned}$$

Comparing the uniform flow depth for these conditions to the critical flow depth shows that $y < y_c$. Therefore, the flow in the channel is *supercritical*.

7.4 OCCURRENCE OF CRITICAL DEPTH

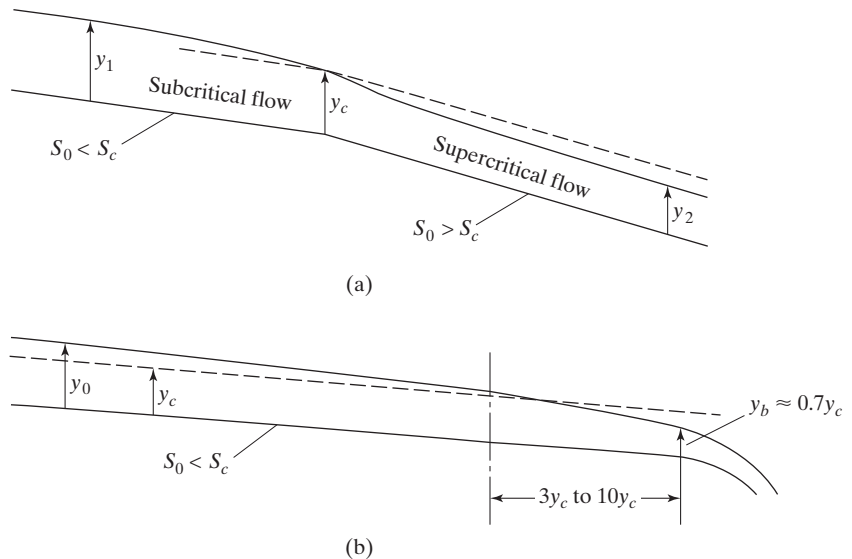
When flow changes from subcritical to supercritical or vice versa, the depth must pass through critical depth. The condition of critical depth implies a unique relationship between y and V or Q . This condition can occur only at a **control section**. In the transition from supercritical to subcritical flow, a **hydraulic jump** will occur, as described in Section 7.8. By measuring the depth at a control section, we can compute a value of Q for a channel based on critical flow equations.

Critical depth occurs when flow passes over a weir or a free outfall with subcritical flow in the channel prior to the control section. Critical depth may also occur in a channel if the bottom is suddenly elevated or if the sidewalls are moved in by contraction. In fact, flumes are designed so as to force flow to pass through critical depth by adjusting the bottom and sides of a channel. In this way, one simply measures the depth in the flume to estimate Q .

In mountainous streams, a sudden change in slope from mild to steep will force the flow condition to pass through critical and will set up the possibility for standing waves or whitewater. One should never design a channel on a slope that is near critical because of the unpredictable water surface. Figure 7-4

Figure 7-4

Occurrence of critical depth.
(a) Change in flow from subcritical to supercritical at a break in slope.
(b) Free outfall.
Mild slope.



indicates two possible conditions under which critical depth can occur in an open channel.

The previous discussion of uniform flow indicates that a channel of constant shape and slope is a requirement for the condition of uniform flow. However, with a natural stream, the shape, size, and slope of the cross section typically varies along the stream length. Such variations produce the condition of nonuniform flow in most floodplain problems of interest to the engineering hydrologist. Equations from uniform flow can be applied to the nonuniform case by dividing the stream into lengths or reaches within which uniform conditions apply.

In an open channel or natural stream the effect of a grade or slope tends to produce a flow with a continually increasing velocity along the flow path. Gravity is opposed by frictional resistance, which increases with velocity, and eventually the two will be in balance, for the case of uniform flow. When the two forces are not in balance, the flow is nonuniform, and is called **gradually varied flow** if changing conditions occur over a long distance. **Rapidly varied flow** occurs when there is an abrupt change or transition confined to a short distance. Rapid flow is covered in more advanced texts that address fluid mechanics and open channel flow (Chow, 1959; Wylie and Streeter, 1978).

Gradually varied flow can occur along the profile of a channel or natural stream, at changes in cross-sectional shape or size, at bends in the stream, and at structures such as dams, bridges, or weirs. Of particular interest in watershed analysis and floodplain hydraulics is the case of the natural stream and the impact associated with bridge or culvert crossings, especially in urban areas. This represents one of the most complex applications of nonuniform flow theory, and computer models have been written to handle all of the necessary computations. An example of a very popular flood profile model is the U.S. Army Corps of Engineers HEC-2 model (Hydrologic Engineering Center, 1982, 1990), which is described in detail later in this chapter (Sections 7.9 to 7.14). HEC-2 is described in more detail in Hoggan (1997) and in the user's manual (HEC, 1990). The new version of HEC-2, called HEC-RAS (HEC, 1995), has been greatly improved with graphical user interface and is described with a detailed example in Section 7.16.

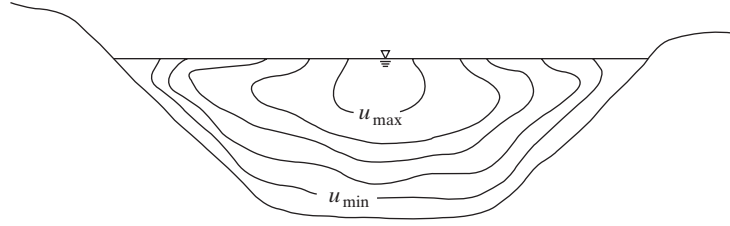
7.5 NONUNIFORM FLOW OR GRADUALLY VARIED FLOW

When flow in an open channel or stream encounters a change in bed slope or a change in cross-sectional shape, the flow depth may change gradually. Such a flow condition where depth and velocity may change along the channel must be analyzed numerically. The energy equation is applied to a differential control volume, and the resulting equation relates change in depth to distance along the flow path. A solution is possible if one assumes that head loss at each section is the same as that for normal flow with the same

7.6 GRADUALLY VARIED FLOW EQUATIONS

Figure 7-5

Typical velocity distribution in an open channel.



velocity and depth of the section. Thus a nonuniform flow problem is approximated by a series of uniform-flow stream segments.

The total energy of a given channel section can be written

$$H = z + y + \frac{\alpha V^2}{2g}, \quad (7-14)$$

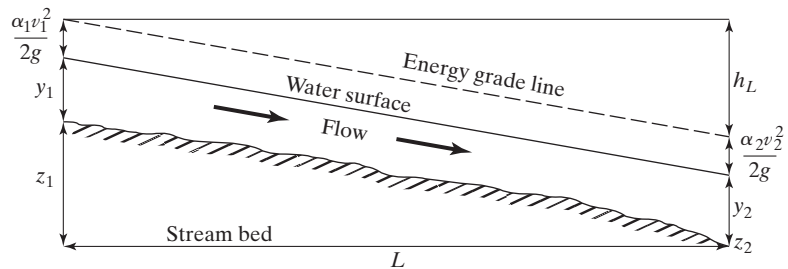
where $z + y$ is the potential energy head above a datum and the kinetic energy head is represented by the velocity head term. The value of α ranges from 1.05 to 1.36 for most channels and is an indication of the velocity distribution across the cross section. It is defined as the energy coefficient, or

$$\alpha = \sum_i \frac{v_i \Delta A}{V^3 A}$$

where v_i is the individual velocity at section ΔA and V is the average velocity over the entire cross section. In many cases, the value of α is assumed to be 1.0 (see Fig. 7-5), but it must be computed for actual streams or rivers where velocity variation can be large.

The energy equation for steady flow between two sections, 1 and 2, a distance L apart becomes (Fig. 7-6)

$$z_1 + y_1 + \frac{\alpha_1 V_1^2}{2g} = z_2 + y_2 + \frac{\alpha_2 V_2^2}{2g} + h_L, \quad (7-15)$$

**Figure 7-6**

Nonuniform flow energy equation. For steady flow between two stations, 1 and 2, a distance L apart.

$$z_1 + y_1 + \frac{\alpha_1 V_1^2}{2g} = z_2 + y_2 + \frac{\alpha_2 V_2^2}{2g} + h_L.$$

where h_L is the head loss from section 1 to section 2. If we assume that $\alpha = 1$, $z_1 - z_2 = S_0 L$, and $h_L = S L$, the energy equation becomes

$$y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + (S - S_0) L. \quad (7-16)$$

The energy slope is determined by assuming that the rate of head loss at a section is the same as that for flow at normal depth with the mean velocity and mean hydraulic radius of the section. Thus, using Manning's equation (ft-s units) and solving for S , we have

$$S = \left(\frac{n V_m}{1.49 R_m^{2/3}} \right)^2, \quad (7-17)$$

where the subscript m refers to a mean value for the reach. If we differentiate Equation (7-14) with respect to x , the distance along the channel, the rate of energy change is found to be

$$\frac{dH}{dx} = \frac{dz}{dx} + \frac{dy}{dx} + \frac{\alpha}{2g} \frac{d(V^2)}{dx}. \quad (7-18)$$

Equation (7-18) describes the variation of water surface profile for gradually varying flows. S_0 and S terms can be substituted. The sign of the slope of the water surface profile depends on whether flow is subcritical or supercritical and on the relative magnitudes of S and S_0 . We observe $V = q/y$ for Equation (7-18), and the last term can be written as ($\alpha = 1$)

$$\frac{1}{2g} \frac{d}{dx} (V^2) = \frac{1}{2g} \frac{d}{dx} \left(\frac{q^2}{y^2} \right) = - \left(\frac{q^2}{g} \right) \left(\frac{1}{y^3} \right) \frac{dy}{dx}. \quad (7-19)$$

Thus,

$$-S = -S_0 + \frac{dy}{dx} \left(1 - \frac{q^2}{gy^3} \right). \quad (7-20)$$

If we include the definition of the Froude number (Fr), then the water surface profile for a rectangular section can be written as

$$\frac{dy}{dx} = \frac{S_0 - S}{1 - (V^2/gy)} = \frac{S_0 - S}{1 - Fr^2}. \quad (7-21)$$

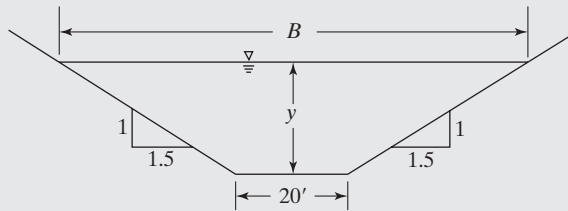
With S_0 and n known and the depths and velocities at both ends of the reach given, the length L of the reach can be computed as follows:

$$L = \frac{[y_1 + (V_1^2/2g)] - [y_2 + (V_2^2/2g)]}{S - S_0}. \quad (7-22)$$

Example 7-5 explores the backwater analysis for a trapezoidal channel and illustrates the water surface profile computation using Equation (7-22). This is referred to as the standard step method, which forms the basis for most floodplain analyses.

EXAMPLE 7-5**WATER SURFACE PROFILE DETERMINATION**

A trapezoidal channel with the dimensions shown in Figure E7-5(a) is laid on a slope of 0.001 ft/ft. The Manning's n value for this channel is 0.025, and the rate of flow through the channel is 1000 cfs. Calculate and plot the water surface profile from the point where the channel ends (assume a free outfall) to the point where $y \leq 0.9y_n$.

Figure E7-5(a)**SOLUTION**

At the point of free outfall, flow will pass through critical depth. Equation (7-13) is used for a nonrectangular channel shape to find the value of critical depth:

$$\frac{Q^2}{g} = \frac{A^3}{B}$$

Referring to the figure, we have

$$\begin{aligned} A &= 20y_c + 2(1/2)(y_c)(1.5y_c) \\ &= y_c(20 + 1.5y_c), \\ B &= 20 + 2(1.5y_c) \\ &= 20 + 3y_c. \end{aligned}$$

Then

$$\frac{(1000 \text{ cfs})^2}{(32.2 \text{ ft/s}^2)} = \frac{y_c^3(20 + 1.5y_c)^3}{(20 + 3y_c)},$$

or

$$\frac{y_c^3(20 + 1.5y_c)^3}{(20 + 3y_c)} = 31,056 \text{ ft}^5.$$

Solving by trial and error or using **Goal Seek** in Excel yields

$$y_c = 3.853 \text{ ft.}$$

The value of normal depth is found using Manning's equation for flow [Eq. (7-3)]:

$$Q = \left(\frac{1.49}{n} \right) AR^{2/3} \sqrt{S_0}.$$

Referring again to the figure, we have

$$P = 20 + 2\sqrt{3.25}y_n$$

$$R = \frac{y_n(20 + 1.5y_n)}{20 + 3.61y_n}.$$

So

$$1000 \text{ cfs} = \frac{1.49}{0.025} [y_n(20 + 1.5y_n)] \left[\frac{y_n(20 + 1.5y_n)}{(20 + 3.61y_n)} \right]^{2/3} \sqrt{.001},$$

or

$$\frac{[y_n(20 + 1.5y_n)]^{5/3}}{(20 + 3.61y_n)^{2/3}} = 530.58.$$

Solving by trial and error or using Goal Seek in Excel yields,

$$y_n = 6.55 \text{ ft}$$

and

$$0.9y_n = 5.90 \text{ ft}.$$

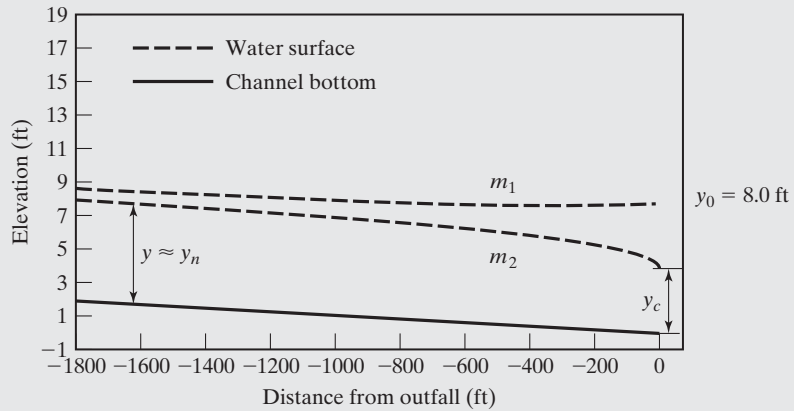
Thus the range of depth for which the profile is desired is 3.85 ft to 5.90 ft. For chosen values of y_1 and y_2 , V_1 and V_2 can be found by $V = Q/A$, and S can be calculated using Equation (7-17). P and R can also be computed from the channel geometry for sections 1 and 2. Thus for each pair of chosen y_1 and y_2 , a reach length $L = \Delta x$ can be determined from Equation (7-22). Values of y , A , P , R , V , V_m , R_m , S , $y + V^2/2g$, Δx , and x are given in Table E7-5, which can be easily solved by an Excel spreadsheet.

The negative sign for x values indicates that the profile is a backwater profile that extends upstream from the boundary condition of $y = y_c$. As

Table E7-5. Backwater Computation

Y (ft)	A (ft ²)	P (ft)	R (ft)	V (ft/s)	V_m (ft/s)	R_m (ft)	S	$y + \frac{V^2}{2g}$ (ft)	Δx (ft)	$x = \sum \Delta x$ (ft)
3.85	99.23	33.88	2.93	10.08	9.71	3.01	0.0061	5.43		0
4.10	107.22	34.78	3.08	9.33	8.94	3.17	0.0048	5.45	-3.92	-3.9
4.40	117.04	35.86	3.26	8.54	8.21	3.35	0.0038	5.53	-21.05	-25.0
4.70	127.14	36.95	3.44	7.87	7.57	3.53	0.0030	5.66	-46.43	-71.4
5.00	137.50	38.03	3.61	7.27	7.01	3.70	0.0024	5.82	-80.00	-151.4
5.30	148.14	39.11	3.79	6.75	6.52	3.87	0.0020	6.01	-135.70	-287.1
5.60	159.04	40.19	3.95	6.29	6.08	4.04	0.0016	6.21	-200.00	-487.1
5.90	170.22	41.27	4.12	5.87	5.69	4.21	0.0013	6.44	-383.30	-870.4
6.20	181.70	42.35	4.29	5.50	5.34	4.37	0.0011	6.67	-766.70	-1637.1
6.50	193.40	43.44	4.45	5.17				6.92	-2500.00	-4137.1

Figure E7-5(b)



seen in column 1, y -values are chosen at a regular interval and extend up to but not beyond the normal depth. It is possible to use smaller intervals for y near the free outfall, since this is the steepest part of the profile and y is changing quickly over small distances of x . The water surface profile for the example is plotted in Figure E7-5(b) with respect to the channel bottom, using the free outfall as the datum.

Problem 7.23 explores the effect of altering the downstream boundary condition to be $y = 8.0$ ft, and the result is also plotted in Figure E7-5(b). Note that the standard step method can be used to evaluate any mild slope profile, given a downstream starting condition, channel geometry, and flow rate.

7.7 CLASSIFICATION OF WATER SURFACE PROFILES

The classification of water surface profiles for nonuniform flow can be studied most easily for rectangular channels. Then Equation (7-21) becomes

$$\frac{dy}{dx} = \frac{S_0 - S}{1 - \text{Fr}^2} = \frac{S_0(1 - S/S_0)}{1 - \text{Fr}^2},$$

where

$$S = \frac{n^2 V^2}{y^{4/3}} = \frac{n^2 Q^2}{b^2 y^{10/3}}$$

in metric units. From Manning's equation for a wide rectangular channel of width b , Q is related to normal depth:

$$Q = \frac{R^{2/3} \sqrt{S_0} A}{n} = \frac{y_n^{2/3} \sqrt{S_0} b y_n}{n}. \quad (7-23)$$

Solving for S_0 , we have

$$S_0 = \frac{n^2 Q^2}{b^2 y_n^{10/3}} \quad (7-24)$$

and

$$S/S_0 = (y_n/y)^{10/3}. \quad (7-25)$$

With the Froude number expressed in terms of critical depth,

$$\text{Fr} = (y_c/y)^{3/2}, \quad (7-26)$$

the slope of the water surface for this case becomes

$$\frac{dy}{dx} = S_0 \left[\frac{1 - (y_n/y)^{10/3}}{1 - (y_c/y)^3} \right]. \quad (7-27)$$

The sign of dy/dx , the water slope, depends on the relations among depths y , y_n , and y_c in Equation (7-27) (see Table 7-2).

From earlier consideration of critical flow, in Section 7.3, the following definitions are presented:

If $y_n > y_c$, then $S_0 < S_c$ and the slope is mild (subcritical).

If $y_n = y_c$, then $S_0 = S_c$ and the slope is critical.

If $y_n < y_c$, then $S_0 > S_c$ and the slope is steep (supercritical).

The three conditions are referred to as mild (*M*), critical (*C*), and steep (*S*) slopes. Other slopes may be **horizontal** (*H*) or **adverse** (*A*). The derivation presented here can also be shown to apply to a general cross-sectional channel.

For a given bed slope, the shape of the water surface profile depends on actual depth y relative to y_n and y_c . If the water surface lies above both normal and critical depth lines, it is type 1; if it lies between these lines, it is type 2; and if it is below both lines, it is type 3. There are a total of twelve possibilities, as shown in Table 7-2. For most cases of interest in the analysis of floodplains or floodways, the *M1* profile is the most important and most observed case.

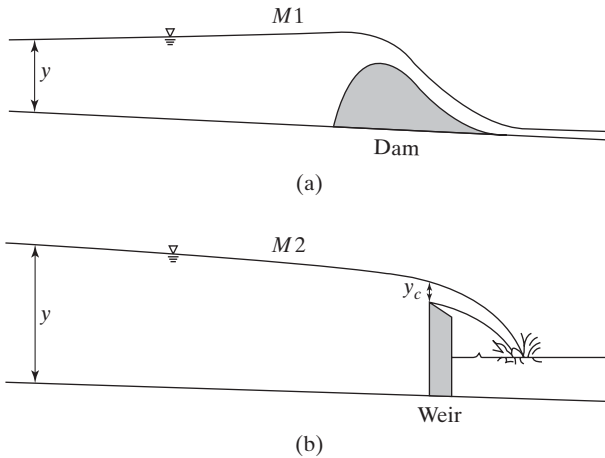
Mild Slope Cases

For the *M1* profile, called the **backwater curve**, $y > y_n$ and $y_n > y_c$, and both the numerator and denominator are positive in Equation (7-27). The water surface slope dy/dx is positive, and as y increases, the water slope approaches S_0 and the free surface approaches the horizontal. As depth increases, velocity must decrease gradually to maintain constant Q . The *M1* profile typically occurs where dams, bridges, or other control structures tend to create a backwater effect along a stream [see Fig. 7-7(a)]. For a uniform, prismatic channel, Equation (7-21) can be solved using the **standard step method** depicted in Example 7-5. Lengths upstream of the dam are calculated incrementally as depths are gradually decreased toward y_n , since the depth y will asymptotically approach normal depth at some point upstream. Accuracy of the resulting calculation depends on the number of increments selected between the starting water surface at the dam and the value of y_n .

Table 7-2. Surface Profiles for Gradually Varied Flow

Surface Profiles	Curve	Depth	Flow	Surface Slope
Mild slope, $S_0 < S_c$ 	M1	$y > y_n > y_c$	Subcritical	Positive
	M2	$y_n > y > y_c$	Subcritical	Negative
	M3	$y_n > y_c > y$	Supercritical	Positive
Steep slope, $S_0 > S_c$ 	S1	$y > y_c > y_n$	Subcritical	Positive
	S2	$y_c > y > y_n$	Supercritical	Negative
	S3	$y_c > y_n > y$	Supercritical	Positive
Critical slope, $S_0 = S_c$ 	C1	$y > y_c = y_n$	Subcritical	Positive
	C3	$y < y_c = y_n$	Supercritical	Positive
Horizontal slope, $S_0 = 0$ 	H2	$y > y_c$	Subcritical	Negative
	H3	$y < y_c$	Supercritical	Positive
Adverse slope, $S_0 < 0$ 	A2	$y > y_c$	Subcritical	Negative
	A3	$y < y_c$	Supercritical	Positive

For a natural stream, cross-sectional areas and slopes may change at various points along the stream. Since Manning's n values may vary both with location in the cross section and with location along the stream, more sophisticated computer programs must be used for calculation of the M1 profile. Sections 7.9 through 7.15 describe in detail the HEC-2 model, one of the most widely used water surface profile models. The model, updated to HEC-RAS

**Figure 7-7**

(a) Example of an M1 profile.
(b) Example of an M2 profile.

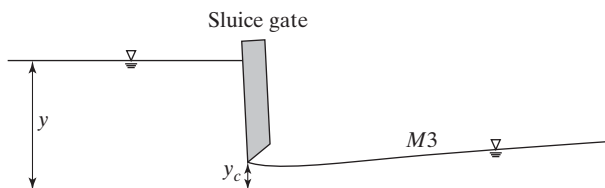
by the Hydrologic Engineering Center in 1995, is primarily used to define floodplains and floodways for natural streams (HEC, 1990, 1995).

For the $M2$ profile, $y_n > y > y_c$ and flow is subcritical, with a negative numerator and positive denominator in Equation (7-27). Thus, dy/dx is negative, and depth decreases in the direction of flow, with a strong curvature when $y \rightarrow y_c$, where the equation predicts $dy/dx \rightarrow \infty$. Thus the equation does not strictly hold at $y = y_c$ and Equation (7-11) applies. The $M2$ curve, called the drawdown curve, can occur upstream from a section where the channel slope changes from mild to critical or supercritical, as in flow over a spillway crest or weir [see Fig. 7-7(b)]. The $M2$ profile is created due to a control condition *downstream*, since flow is subcritical.

The $M3$ profile is for supercritical flow where $y_n > y_c > y$; both numerator and denominator are negative, so dy/dx is positive, and depth increases in the direction of flow. The $M3$ curve occurs downstream of a spillway or sluice gate, and as critical depth is approached from below, a sudden transition called the hydraulic jump occurs from supercritical to subcritical conditions (see Figs. 7-8 and 7-9).

Steep Slope Cases

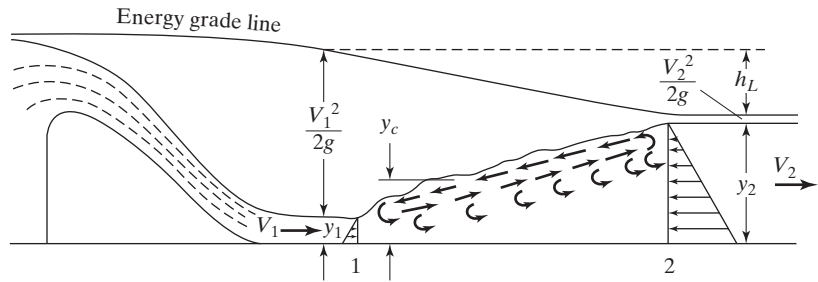
The steep curves that occur on steep slopes ($S_0 > S_c$), where $y_c > y_n$, can be analyzed in the same way as the M curves, with the change that downstream control occurs for subcritical flow ($S1$) and upstream control occurs for supercritical flow ($S2$ and $S3$). For example, a dam on a steep slope will

**Figure 7-8**

Example of an M3 profile.

Figure 7-9

Hydraulic jump on horizontal bed following spillway.



produce an $S1$ curve, which is preceded by a hydraulic jump. A sluice gate on a steep channel will produce an $S3$ curve, which will smoothly approach the uniform-depth line ($y = y_n$). Table 7-2 illustrates the steep slope cases.

Horizontal and Adverse Slope Cases

The $H2$ and $H3$ profiles correspond to the $M2$ and $M3$ curves for $S_0 = 0$. $H1$ cannot exist, since for $S_0 = 0$, normal depth is infinite. The profile slope can be determined from Equation (7-21), but this profile can exist only for a short section contained in more complex channel reaches, since flow cannot continue indefinitely on a horizontal bed.

The adverse slope, $S_0 < 0$, occurs where flows sometimes move against gravity; as in the case of horizontal slopes, the adverse slope can occur only for a short section in a more complex channel system. In some natural channel systems, there are occasional reaches where these conditions can and do occur.

7.8 HYDRAULIC JUMP

The hydraulic jump is covered in detail in most fluid mechanics textbooks (Daugherty et al., 1985; Fox and McDonald, 1985). The transition from supercritical to subcritical flow produces a marked discontinuity in the surface, characterized by a steep upward slope of the profile and a significant loss of energy through turbulence. The hydraulic jump can best be explained by reference to the $M3$ curve of Table 7-2 and Figure 7-8, downstream of a sluice gate. The flow decelerates because the mild slope is not great enough to maintain constant supercritical flow, and the specific energy E decreases as the depth increases. When downstream conditions require a change to subcritical flow, the need for the change cannot be telegraphed upstream. Theory calls for a vertical slope of the water surface. Therefore, a hydraulic jump occurs and flow changes from supercritical to subcritical conditions.

When flow at a section is supercritical and downstream conditions require a change to subcritical flow, the abrupt change in depth involves a significant energy loss through turbulent mixing. A jump will form when slope changes from steep to mild, as on the apron at the base of a spillway. In fact, stilling basins below spillways are designed to dissipate the damaging energy of supercritical velocities, and downstream erosion is held in check.

An equation can be derived for horizontal or very mild slopes to relate depth before and after the jump has occurred. Assuming hydrostatic pressure

and neglecting gravity and friction forces in this case, Newton's equation for momentum change becomes (Fig. 7-9)

$$\sum F_x = \gamma h_1 A_1 - \gamma h_2 A_2 = (\gamma Q/g)(V_2 - V_1), \quad (7-28)$$

For a rectangular channel of unit width,

$$\frac{V_1^2 y_1}{g} + \frac{y_1^2}{2} = \frac{V_2^2 y_2}{g} + \frac{y_2^2}{2}.$$

The continuity equation implies, for Sections 1 and 2, that

$$V_1 y_1 = V_2 y_2. \quad (7-29)$$

Solving Eqs. (7-28) and (7-29) simultaneously after eliminating V_2 and rearranging,

$$y_2^2 - y_1^2 = \left(\frac{2V_1^2 y_1}{g} \right) \left(\frac{y_2 - y_1}{y_2} \right). \quad (7-30)$$

Dividing by $y_2 - y_1$, multiplying by y_2/y_1^2 and solving for y_2/y_1 , using the quadratic formula, we obtain

$$y_2/y_1 = \frac{\sqrt{1 + 8 \text{Fr}_1^2} - 1}{2}, \quad (7-31)$$

where Fr_1 is the upstream Froude number. Thus the ratio y_2/y_1 is dependent only on upstream Froude number Fr_1 , and y_1 and y_2 are called conjugate depths. An increase in depth requires $\text{Fr}_1 > 1$, or supercritical conditions upstream. The head loss and location of a hydraulic jump can also be calculated, but this is beyond the scope of the present coverage. Example 7-6 illustrates the use of Equation (7-31) to evaluate depths for the hydraulic jump.

CALCULATION OF THE DEPTH OF A HYDRAULIC JUMP

EXAMPLE 7-6

A sluice gate is constructed across an open channel. When the sluice gate is open, water flows under it, creating a hydraulic jump, as shown in Figure E7-6. Determine the depth just downstream of the jump (point b) if the depth of flow at point a is 0.0563 m and the velocity at point a is 5.33 m/s.

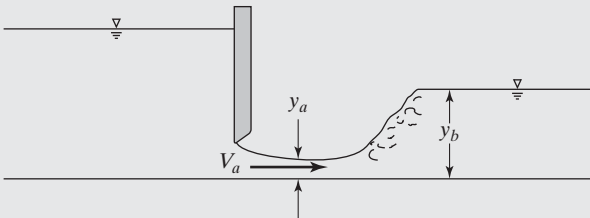


Figure E7-6

SOLUTION

Equation (7-31) gives

$$\frac{y_b}{y_a} = \frac{(\sqrt{1 + 8 \text{Fr}_a^2}) - 1}{2}.$$

The upstream Froude number is found from

$$\begin{aligned} \text{Fr}_a &= \frac{V_a}{\sqrt{gy_a}} \\ &= \frac{5.33 \text{ m/s}}{\sqrt{(9.81 \text{ m/s}^2)(0.563 \text{ m})}}, \\ \text{Fr}_a &= 7.17. \end{aligned}$$

Then

$$\begin{aligned} y_b &= [(\sqrt{1 + (8)(7.17)^2}) - 1][(0.563 \text{ m})/2] \\ y_b &= 0.544 \text{ m}. \end{aligned}$$

7.9
INTRODUCTION
TO THE HEC-RAS
MODEL

The HEC-2 model was originally developed in the 1970s by the Hydrologic Engineering Center (1982) of the U.S. ACOE. The program was updated to HEC-RAS in 1995 with an easy-to-use graphical user interface (GUI). The program is designed to calculate water surface profiles for steady, gradually varied flow in natural or man-made channels. The computational procedure is based on solution of the one-dimensional energy equation [Eq. (7-16)] using the standard step method. The program can be applied to floodplain management and flood insurance studies to evaluate **floodway encroachments** and to delineate **flood hazard** zones. The model can also be used to evaluate effects on water surface profiles of channel improvements and levees, altered flow values, as well as the presence of bridges or other structures in the floodplain.

The main objective of the HEC-RAS program is to compute water surface elevations at cross-section locations of interest along a river or stream, for given flow values. Data requirements include flow regime, starting elevation, flow rate, loss coefficients, roughness, cross-sectional geometry, and reach lengths. Profile computations begin at a cross section with known or assumed starting conditions and proceed upstream for subcritical flow or downstream for supercritical flow. Subcritical profiles are constrained to critical depth or above, and supercritical profiles are constrained to critical depth or below. The program will not allow profile computations to cross critical depth in most cases because the governing equations do not apply for $y = y_c$, as discussed in Section 7.4.

The HEC-RAS program is often used in association with the HEC-HMS program (Chapter 5) for determination of flood flows and flood elevations in a particular watershed. Peak flows at various locations along the main stream or channel are computed by HEC-HMS for a given rainfall.

These peak flows are then used in HEC-RAS to calculate the steady-state, nonuniform water surface profile along the stream. For example, the 100-yr rainfall could be used in HEC-HMS to calculate 100-yr flows, which then could be used in HEC-RAS to predict the 100-yr floodplain. RAS can be used to compute storage–outflow relationships to be used in HEC-HMS by computing a series of water surface profiles and corresponding discharges.

HEC-RAS is a very sophisticated computer program designed to handle a number of hydraulic computations in a single run in a GUI. The main focus in the design of the interface was to make it easy to use the software, while still maintaining a high level of efficiency for the user. The basic program and the basic input data requirements are relatively easy to learn and are covered in detail in the sections that follow. Special features and other options are described in the user's manual (Hydrologic Engineering Center, 1995, 2000, 2001, 2002), the book of floodplain modeling using HEC-RAS (Haestad et al., 2003), and an earlier text describing HEC-2 (Hoggan, 1997).

Equations presented in Section 7.6 are solved using the standard step method to compute an unknown water surface elevation at a particular cross section. The next several sections (7.10 to 7.14) present the theory behind the HEC-RAS model for natural or man-made streams. The new release of HEC-RAS is presented in detail with an example in Section 7.14. HEC-RAS is capable of performing one-dimensional water surface profile calculations for steady, gradually varied flow in natural or constructed channels. The following two equations are solved for basic water surface profiles by an iterative procedure for upstream (subscript 2) and downstream (subscript 1) sections:

7.10 THEORETICAL BASIS FOR HEC-RAS

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} + h_e, \quad (7-32)$$

$$h_e = L\bar{S}_f + C \left(\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right), \quad (7-33)$$

where

- WS_1, WS_2 = water surface elevations at ends of reach,
- V_1, V_2 = mean velocities (total discharge/total flow area) at ends of reach,
- α_1, α_2 = velocity or energy coefficients for flow at ends of reach,
- g = gravitational constant,
- h_e = energy head loss,
- L = discharge-weighted reach length,
- $\bar{S}_f$ = representative friction slope for reach,
- C = expansion or contraction loss coefficient.

Cross-section data are traditionally defined looking in the downstream direction. The discharge-weighted reach length L is computed by weighting lengths in the **left overbank** (LOB), **channel** (CH), and **right overbank** (ROB) with their respective flows at the end of the reach. A representative

friction slope is usually expressed as follows, although alternative equations can be used:

$$\bar{S}_f = \left(\frac{Q_1 + Q_2}{K_1 + K_2} \right)^2, \quad (7-34)$$

where K_1 and K_2 represent the conveyance at the beginning and end of a reach. **Conveyance** is defined from Manning's equation (U.S. customary units) as

$$K = \frac{1.49}{n} AR^{2/3}. \quad (7-35)$$

The total conveyance for a cross section is obtained by summing the conveyance from the left and right overbanks and the channel. The energy or velocity coefficient α is obtained with the equation

$$\alpha = \left(\frac{A_T^2}{K_T^3} \right) \left(\frac{K_{LOB}^3}{A_{LOB}^2} + \frac{K_{CH}^3}{A_{CH}^2} + \frac{K_{ROB}^3}{A_{ROB}^2} \right), \quad (7-36)$$

where the subscript T is for cross-sectional total, LOB is for left overbank, CH is for channel, and ROB is for right overbank.

The computational procedure for the iterative solution of Eqs. (7-32) and (7-33) is as follows:

1. Assume a water surface elevation at the upstream cross section (i.e., a first trial is $\Delta WS = (Q/K)^2 L$).
2. Based on assumed elevation, determine the corresponding total conveyance and velocity head for the upstream section (2).
3. With values from step 2, compute friction slope $\bar{S}_f$ and solve Equation (7-33) for head loss h_e .
4. With values from steps 2 and 3, solve Equation (7-32) for WS_2 .
5. Compare the computed value of WS_2 with the values assumed in step 1, and repeat steps 1–5 until values agree to within 0.01 ft (0.01 m).

The first iterative trial is based on the friction slope from the previous two cross sections. The second trial is an average of the computed and assumed elevations from the first trial. Once a balanced water surface elevation has been obtained for a cross section, checks are made to be sure that the elevation is on the correct side of the critical water surface elevation. If otherwise, critical depth is assumed and a message to that effect is provided. The occurrence of critical depth in the program is usually the result of a problem with reach lengths or flow areas unless a critical flow condition actually occurs.

The following assumptions are implicit in the equations and procedures used in the program: (1) flow is steady, (2) flow is gradually varied, (3) flow is one dimensional, and (4) river channels have small slopes (less than 1:10). If any of these assumptions are violated, the results from the HEC-RAS program may be in error.

Flow is assumed to be steady because time-dependent variables are not included in the energy equation, Equation (7-32). When flow conditions do

vary with time at a discrete location, the flow is classified as **unsteady**. Since the equations used to model unsteady-state flow are quite complex, civil engineers usually prefer to assume steady flow conditions when computing peak water surface elevations. However, engineers sometimes encounter situations for which the assumption of gradually varied, steady-state flow is not appropriate and they must turn to unsteady-state flow modeling. Generally, the St. Venant equations, a combination of both the continuity and momentum equations, are used to handle unsteady-state flow problems (Chapter 4). Since it is not possible to solve the full St. Venant equations analytically, this difficulty leads to a number of simplifications such as neglecting the local acceleration term, convective acceleration term, and pressure term when using backwater methods, Muskingum/Muskingum Cunge methods, and kinematic wave routing, respectively. By doing so, simplified representation of St. Venant equation can be obtained to solve for stage and flow approximately. The Unsteady NET work simulator (UNET) was modified for HEC-RAS as a solver to simulate one-dimensional unsteady flow through a network of open channels (Barkau, 1992). Detailed features and other information about unsteady-state modeling are described in the user's manual (Hydrologic Engineering Center, 2002) and in the book of floodplain modeling using HEC-RAS (Haestad et al., 2003).

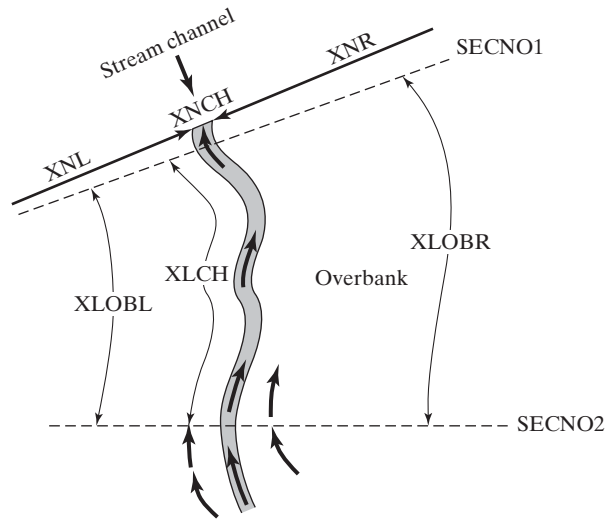
An overview of the cross-sectional data required of the HEC-RAS model is shown in Figure 7–10a along with a typical plan view of a floodplain (Fig. 7–10b). The model begins its computation with a water surface elevation and a beginning discharge at the most downstream cross section for a mild slope case; near the mouth of a stream or river steep slopes are computed from upstream to downstream. The orientation for left or right overbank is for the user to be standing in the stream looking downstream in the direction of flow. Discharges are specified at every cross section from upstream to downstream for each reach, as HEC-RAS performs calculations in an upstream direction for sub-critical flows. Once one flow value is entered at a specific cross section, it is assumed that the flow remains constant until another flow value is encountered at downstream location. Input lateral inflows can be accommodated. Cross-sectional geometry is specified in terms of ground surface profiles oriented perpendicular to the stream and the measured distances between them in an upstream direction. Spacing of cross sections is dependent on a number of variables, described below. Within each cross section, the left overbank, the right overbank, and the channel locations must be specified (Fig. 7–11).

Loss coefficients are of great importance to the hydraulic computations performed by HEC-RAS; in particular, Manning's roughness factor n must be specified for the channel and overbank areas of the stream, and may vary both horizontally and/or vertically. Contraction or expansion coefficients due to changes in channel cross section must also be specified, especially around bridges and other structures. Figure 7–11 indicates how a typical cross section is specified.

Once the basic data requirements and flow values have been met for HEC-RAS, execution of the program will provide water surface elevations,

7.11 BASIC DATA REQUIREMENTS (STEADY STATE)

Figure 7-10a
HEC-RAS cross-section parameters.



----- Centerline of channel

----- Cross section

----- Channel bank

→ → → Direction of flow

XNL = Manning's n value for left overbank

XNR = Manning's n value for right overbank

XNCH = Manning's n value for channel

XLOBL = Distance along flow path of left overbank between sections

XLOBR = Distance along flow path of right overbank between sections

XLCH = Distance along channel centerline between sections

SECNO1 = First downstream cross section

SECNO2 = Cross section just upstream of SECNO1

velocities, areas, volumes, top widths, and other pertinent geometric data for each cross section. Output can be organized into tables for easy reference or can be plotted as cross sections or longitudinal profiles along the stream. Thus HEC-RAS uses hydraulic and geometric data for a stream and peak flow values to arrive at estimates of the resulting water surface at each section under steady-state flow condition.

Steady flow data are required in order to perform a steady water surface profile calculation. Input data consist of: flow regime, boundary conditions, and peak discharge information. The flow regime (subcritical, supercritical, or mixed flow regime) is specified on the Steady Flow Analysis window of the user interface. Boundary conditions are necessary to establish the starting water surface at the ends of the river system (upstream or downstream) in order for the program to begin the calculations. In a subcritical flow regime, boundary conditions are necessary only at the downstream ends of the river system; if a supercritical flow regime is to be calculated, boundary conditions are necessary only at the upstream ends. If a mixed flow

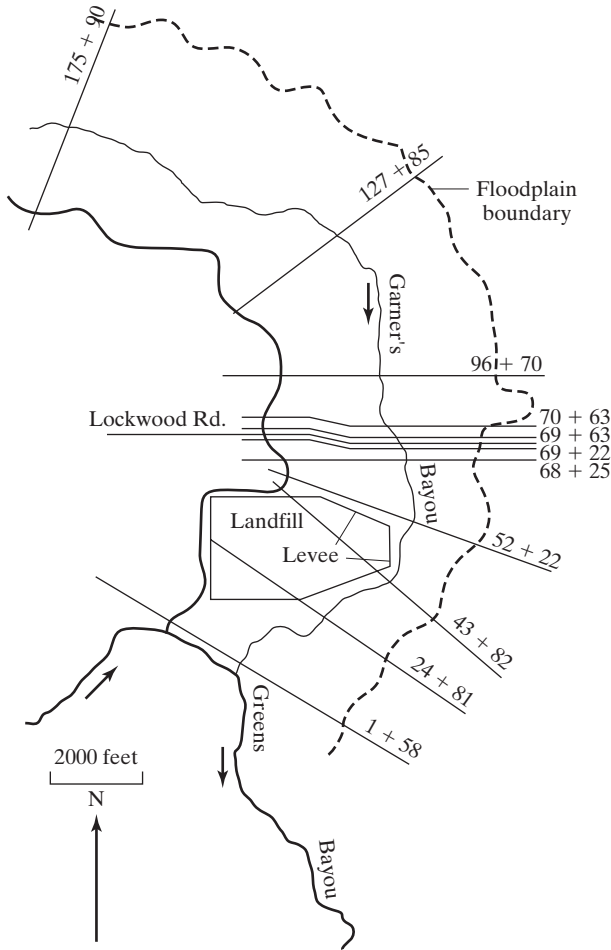


Figure 7-10b
Typical cross-section layout for floodplain analysis.

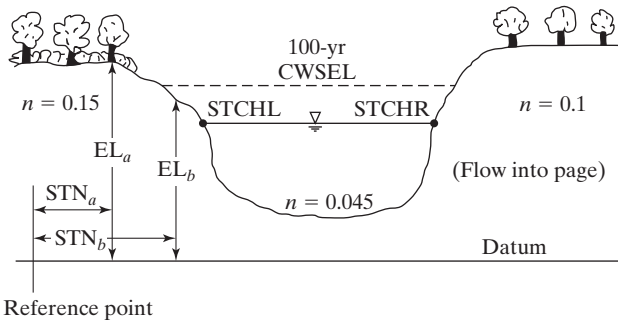


Figure 7-11
Typical cross section.

EL_a, EL_b = Ground elevation above datum for points a and b , respectively
 STN_a, STN_b = Distance from reference point for points a and b , respectively
 STCHL = Left channel bank station (when looking downstream)
 STCHR = Right channel bank station (when looking downstream)
 Note: Up to 100 points (EL, STN) may be defined for each section.

regime calculation is to be performed, then boundary conditions must be entered at both ends of the river system.

Four types of boundary conditions are available to the user to define starting water surface elevations: (1) known water surface elevation, (2) critical depth, (3) normal depth, or (4) rating curves. The user must enter a known water surface elevation for each profile to be computed. The critical depth condition should be used at locations where critical conditions are known to exist, such as at a waterfall, weir, or rapids. The normal depth will be calculated for each profile based on a user-entered energy slope. For each profile to be computed, the elevation is interpolated from the rating curve given the flow, using linear interpolation between the user-entered points. The user is allowed to choose only one boundary condition from above four options in the HEC-RAS program for each profile. (See Section 7.14.)

Several types of loss coefficients are utilized by the program, such as Manning's n for friction loss, contraction and expansion loss, and bridge loss coefficients for weir or pressure flow. These values are typically indicated on Cross Section Data Editor window interface in the HEC-RAS program. Selection of an appropriate value for Manning's n is very significant to the accuracy of the computed water surface profiles. Typical values of Manning's n for various channels were given in Table 7-1.

Contraction or expansion of flow due to changes in the channel cross section is a common cause of energy losses within a reach. These losses can be computed by specifying contraction and expansion coefficients. For small changes in river cross section, coefficients are typically 0.1 and 0.3 for contraction and expansion, respectively, up to 0.3 and 0.5 at bridges. For abrupt changes, the coefficients may be as high as 0.6 and 1.0. The coefficients can be changed at any cross section shown in the Cross Section Data Editor (see Section 7.14).

Cross-sectional geometry for natural streams is specified in terms of ground surface profiles (cross sections) and the measured distances between them (reach lengths). Generally, cross sections should cover the entire floodplain and should be perpendicular to the main flow line (Fig. 7-10b). Cross sections are required at representative locations along a stream length, and where abrupt changes occur, several cross sections should be used, as in the case of a bridge. Cross-section spacing is a function of stream size, slope, and the number of abrupt changes.

Each cross section is identified and described as a unique river station number. Each data point is given a station number corresponding to the horizontal distance from a zero point on the left. The elevation and corresponding station number of each data point are input into the Cross Section Data Editor window, with a maximum of 500 points allowed. It is important that the cross section be **oriented looking downstream** so that the lowest station numbers are on the left. The left- and right-bank station numbers separating the main channel from the overbank areas must be specified on the Cross Section Data Editor window. Endpoints that are too low will automatically be extended vertically and noted in the output. A number of program options are available

to easily add or modify cross-sectional data. For example, a cross section can be easily duplicated or adjusted vertically or horizontally by changing variables in the Cross Section Data Editor. Channel improvement or encroachment options are slightly more difficult to implement and are described later.

The last basic data requirement involves the definition of reach lengths for the left overbank, right overbank, and the channel. Channel reach lengths are typically measured along the curvature of the stream profile (thalweg), and overbank reach lengths are measured along the center of mass of overbank flow (see Fig. 7–10b).

The HEC-RAS model has been designed with maximum flexibility so that the user has a number of optional capabilities. All special options are described in the user's manual, but only those most commonly used will be discussed here (Hydrologic Engineering Center, 2000; Haestad et al., 2003). In this section the main options described are multiple profile analysis, bridge losses, encroachment, channel improvement, and storage outflow data generation. Options not described here include critical depth, effective flow area, friction loss equations, interpolated cross sections, tributary stream profiles, Manning's n calculation, split-flow option, and ice-covered streams (see the HEC-RAS user's manual).

HEC-RAS can compute up to 500 profiles in a single run for the same geometric data within a single execution of the steady flow computations. Typically, this option allows the user to evaluate flows of different magnitudes (2, 5, 10, 25, 100, 500 yr return periods) in a single run. The multiple-profile option also allows for the generation of stream routing data for the modified Puls method in HEC-HMS. For this purpose, storage–outflow data in a tabular form are automatically created and stored in a Data Storage System (DSS), which is able to be accessed by the HEC-RAS program. Data storage is accomplished through the use of ASCII and binary files as well as the HEC-DSS. Input data are stored in ASCII and binary files under separate categories of project, plan, geometry, steady flow, unsteady flow, and sediment data. Output data are predominantly stored in separate binary files. Data can be transferred between HEC-RAS and other programs by utilizing HEC-DSS (Hydrologic Engineering Center, 2002).

Energy losses through culverts and bridges are computed in two steps. First, losses due to expansion and contraction upstream and downstream of the structure are computed. Second, the HEC-RAS bridge method is usually used to compute the loss through the structure itself. The bridge method should be used for bridges with piers with low-flow controls, for pressure flow, and when flow passes through critical depth within the structure. The bridge method allows for both pressure flow and weir flow or any combination of the two. HEC-RAS bridge methods are described in detail in Section 7.13. Chapter 9 presents a detailed culvert example from HEC-RAS.

7.12 OPTIONAL HEC- RAS CAPABILITIES

A number of methods are available in HEC-RAS for specifying encroachments for floodway studies. The floodway is defined as the area that passes the 100-yr flow where encroachments (i.e. levees) increase the water surface by 1 ft. Stations and elevations of the left or right encroachment can be specified for individual cross sections using method 1. Method 4 requires an equal loss of conveyance on each side of the channel, and each modified cross section will have the same discharge as the natural cross section. Figure 12–2 in Chapter 12 shows the floodway compared to the floodplain.

Cross-sectional data may be modified automatically by the channel modification option in HEC-RAS to analyze improvements made to natural stream sections. Trapezoidal excavation is assumed, and geometric data are provided to specify the location, elevation, roughness, side slopes, and bottom width. Numerous program options are available for the user to add or modify cross-section data. One useful feature of the model is the insertion of additional cross sections between those specified by input. For example, when the user wants to duplicate a surveyed cross section, an option from the Cross Section Data Editor window enables him/her to make a copy of any cross-section data. Once a cross section is copied, other options are available to allow the user to modify the horizontal and vertical dimensions of the duplicated cross-section data. Note that the graphical user interface (GUI) for HEC-RAS has revolutionized our ability to handle large-watershed models and to link the analysis to other models within a GIS framework.

7.13
BRIDGE
MODELING IN
HEC-RAS

HEC-RAS program computes energy losses caused by structures such as bridges and culverts in three parts: loss from contraction in the stream immediately upstream from the structure, loss at the structure itself, and loss from the expansion in the stream immediately downstream from the structure. The bridge routines in the HEC-RAS allow users to analyze a bridge with several different methods without changing the bridge geometry. The bridge routines have the capabilities to model low flow (Class A, B, and C), low flow and weir flow (with adjustments for submergence on the weir), pressure flow (orifice and sluice gate equations), pressure and weir flow, and highly submerged flows. The advantage of the HEC-RAS program is that it will automatically switch to the energy equation when the flow over the road is highly submerged. The general bridge layout for cross sections is shown in Figure 7–12.

Bridge Modeling Routines

The bridge modeling routines in HEC-RAS use specific hydraulic equations for a number of possible flow conditions, including **low flow**, **pressure flow**, **weir flow**, or any possible combination (Fig. 7–13). In this method, the opening under the bridge is approximated by a trapezoid with given bottom elevation, bottom width, and side slopes. The presence of piers is accounted for by specifying a total width of flow obstruction due to the piers.

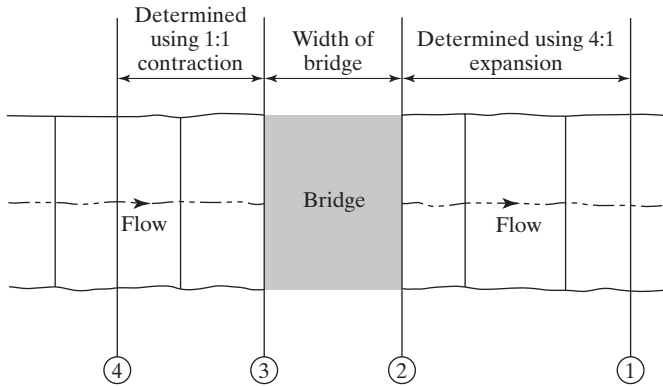


Figure 7-12
Cross section
layout for bridge
modeling.

For low flow under bridges with piers, HEC-RAS considers three conditions, labeled class A, class B, and class C. Class A low flow assumes that the flow is subcritical, and the Yarnell equation is used to compute the change in water surface elevation due to the bridge:

$$H_3 = 2K(K + 10w - 0.6) (a + 15a^4) (V_2^2/2g), \quad (7-37)$$

where

H_3 = change in water surface elevation through bridge (from Section 3 to Section 2),

K = pier shape coefficient,

w = ratio of velocity head to depth downstream of bridge at Section 2,

$a = \frac{\text{obstructed area}}{\text{total unobstructed area}} \quad (\text{at Section 2})$

V_2 = velocity downstream from the bridge at Section 2.

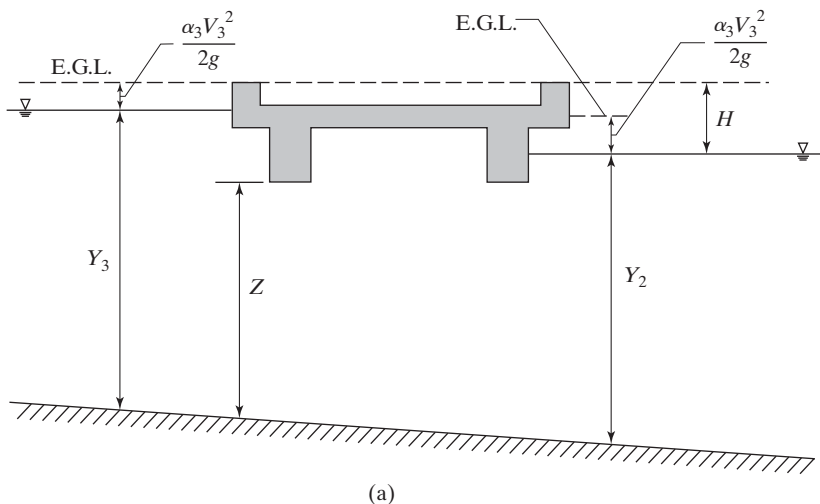
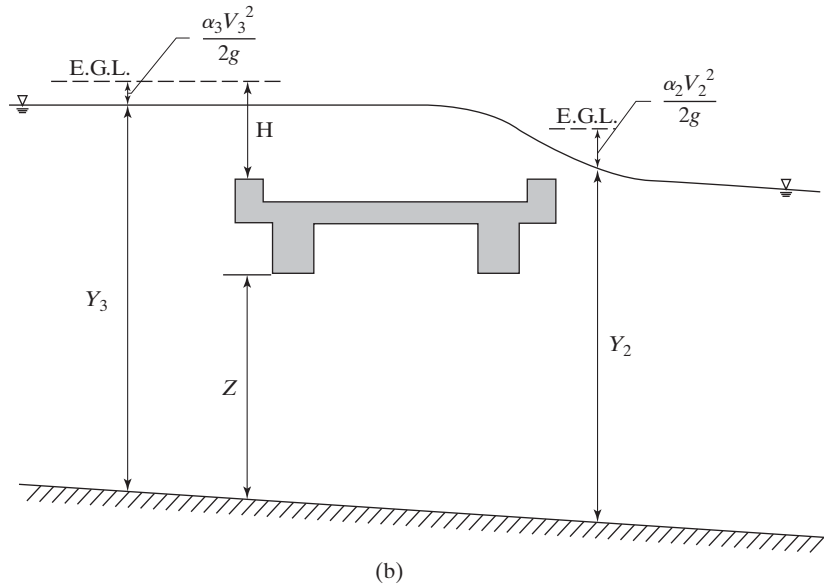


Figure 7-13
Pressure flow and
weir flow through
bridges.

Figure 7-13b
(Continued)



The value of H_3 is added to the downstream water surface elevation after the computation to account for the bridge.

Class B low flow occurs when the water surface profile passes through critical depth underneath the bridge. HEC-RAS uses a momentum balance for cross sections adjacent to and under the bridge, as described in more detail in the HEC-RAS user's manual (Hydrologic Engineering Center, 1995, 2002) and the book of floodplain modeling using HEC-RAS (Haestad et al., 2003). Class C low flow occurs when the flow condition is supercritical, and the same procedure used for class B is applied in this case.

Pressure flow occurs when the bridge deck becomes submerged such that the low chord is in contact with water and a head buildup occurs on the upstream side of the bridge. The energy-based method is applied to handle the pressure flow scenario. Occasionally, the water surface inside of the bridge represents the hydraulic gradeline elevation, not necessarily the actual water surface elevation. When both the upstream and downstream side of a bridge are submerged, we can regard this kind of pressure flow through the bridge as orifice flow in fluid mechanics and can describe it by the equation

$$Q = C_d A (2gH)^{0.5}, \quad (7-38)$$

where

H = total energy difference upstream and downstream,

C_d = discharge coefficient (0.7 to 0.8),

A = cross-sectional area of the bridge opening,

Q = total orifice flow.

HEC-RAS defines H as the distance from the energy grade line to the centroid of the orifice area.

Weir flow occurs when water begins to flow over the bridge and elevated roadway approaches, and HEC-RAS uses the standard weir equation for this flow condition:

$$Q = CLH^{3/2}, \quad (7-39)$$

where

C = weir discharge coefficient,

L = effective length of weir,

H = total energy difference upstream of the bridge and top of the roadway,

Q = flow over the weir.

The coefficient of discharge C varies according to the configuration of the bridge and the roadway and is reduced in HEC-RAS model whenever the weir or bridge is submerged by high tailwater. Typically, discharge coefficient value C ranges from 2.5 to 3.1 for broad-crested weirs when no tailwater effect occurs. For high tailwater elevation, HEC-RAS is able to automatically reduce the amount of weir flow to account for the submergence on the weir. Submergence is defined as a ratio of the water depth above the minimum weir elevation on the downstream side divided by the height of the energy gradeline above the minimum weir elevation on the upstream side. When the weir is highly submerged, HEC-RAS program automatically switches to calculating the upstream water surface by the energy equation instead of using the pressure and weir flow equation. The user can set up the criteria for when to switch to energy-based computations.

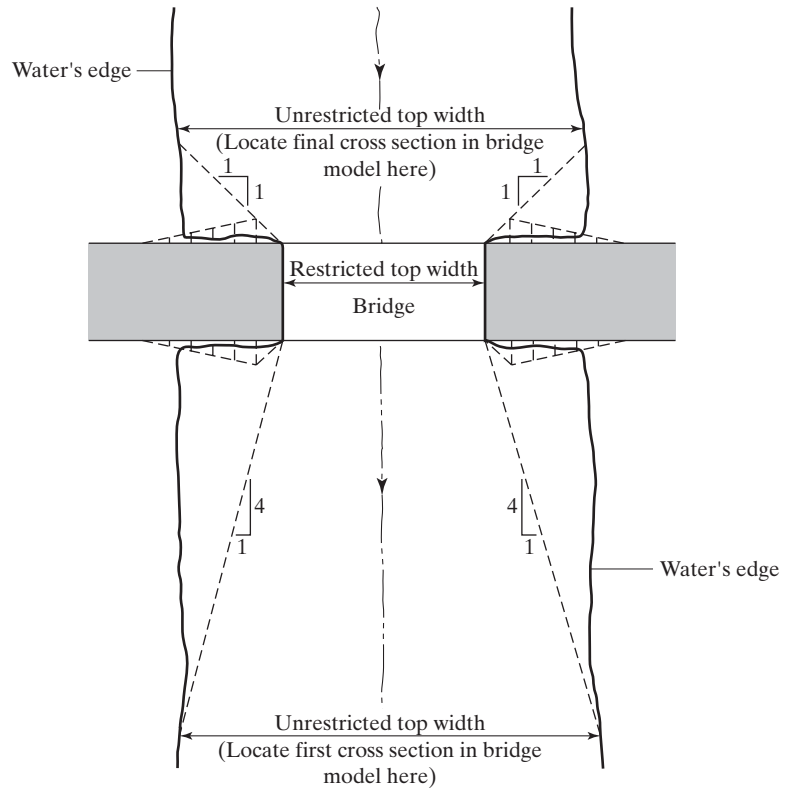
Sometimes combination of low flows or pressure flow occurs with weir flow. Such flows are handled by HEC-RAS using an iterative procedure to balance energy elevations and computed flow rate.

Cross-Section Layout for Bridge Modeling

Figure 7-12 illustrates the cross-section requirements for bridge modeling. In order to perform computation properly, HEC-RAS program requires four key cross sections: (1) one cross section downstream of the bridge where flow is not affected by the bridge, (2) one just downstream of the bridge, (3) one just upstream of the bridge, and (4) one cross section located far enough upstream to be out of the backwater influence. Based on these four chosen cross sections, HEC-RAS program automatically adds two more cross sections immediately inside the upstream and downstream bridge faces. Thus, in HEC-RAS, bridge modeling is usually performed with a total of six cross sections, with four sections provided by the user and two more developed by the program.

The spacing of cross sections downstream of the bridge should be based on a 4:1 expansion of flow, which corresponds to 1 ft laterally for every 4 ft traveled in the flow direction. This phenomenon is shown in Figure 7-14.

Figure 7-14
Expansion and
contraction of
flow at bridges.



The location of the final bridge cross section in the bridge method should be based on a 1:1 contraction of flow. To avoid complications where reach lengths become too long upstream or downstream of bridges, the expansion and contraction criteria should be used to limit the lateral extent of any intermediate (inserted) cross sections.

Ineffective Area Option

Effective flow area is that portion of the total cross-sectional area where flow velocity is normal to the cross section in the downstream direction. Any other areas that do not convey flow in the downstream direction are called **ineffective flow areas**. Figure 7-15 illustrates some examples of ineffective flow areas created by the presence of bridges.

The ineffective flow area may be eliminated from consideration by defining the top of road and low chord profiles of a bridge. The ineffective area option in HEC-RAS may be used to “block out” portions of the channel cross section that are not effective in conveying flow downstream. For example, all flow can be restricted to a channel cross section until the computed water surface elevation exceeds the elevation of one or both channel banks. Encroachment stations for a cross section can be considered using

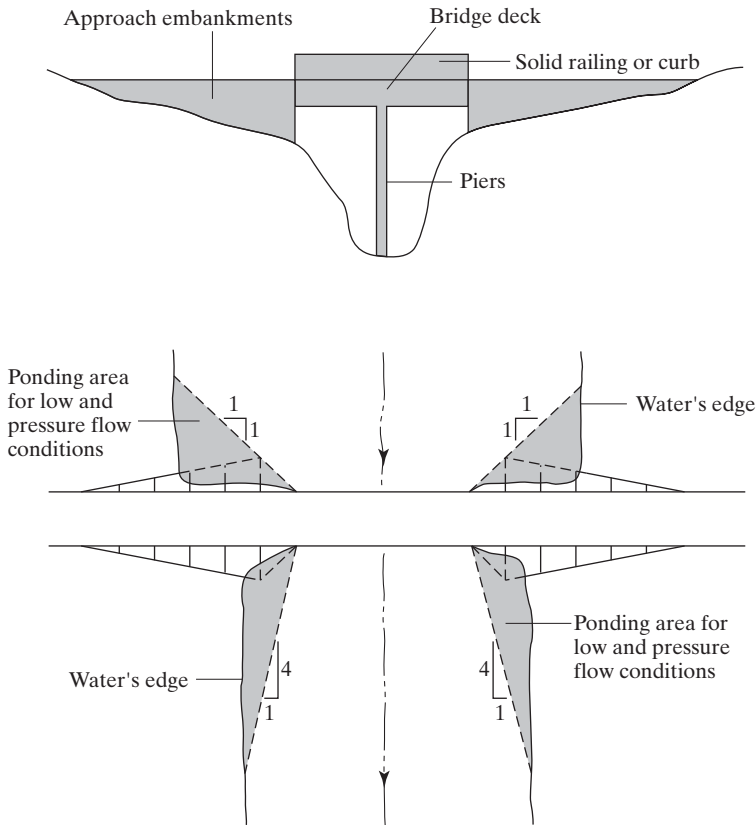


Figure 7-15
Examples of ineffective areas at bridges.

this same method. All parameters necessary for implementing the ineffective area option are contained in the Cross Section Data Editor window.

Detailed discussion of applications of the ineffective area option, together with coefficients for bridge and culvert modeling, are contained in the HEC-RAS user's manual (Hydrologic Engineering Center, 2000) and in books on floodplain modeling using HEC-RAS (Haestad et al., 2003) and in Hoggan (1997).

After the HEC-RAS program is executed for a stream reach, a detailed output for each profile appears. Summary tables provide the basic information to define water surface elevation (WSEL), channel velocity (VCH), area, flow rate (Q), top width, and a host of other variables that could be requested. HEC-RAS could also be used to evaluate

1. effects of altered (increased or decreased) peak flows from urban development or reservoir storage,
2. effects of additional bridge crossings or altered bridge openings,
3. effects of altering roughness coefficients or channel sizes, and
4. effects of changing the starting water surface elevation downstream (i.e., at the confluence with a larger stream).

Finally, HEC-RAS can be used to generate a series of water surface elevations and the corresponding flows to generate a storage–discharge relation for HEC-HMS flood routing. Thus, HEC-HMS and HEC-RAS together provide powerful computer tools for hydrologic analysis and design. More details can be found in Hoggan (1997) and the HEC user manuals, and a detailed example is provided at the end of this chapter.

7.14
HEC-RAS
FEATURES

The impetus behind the creation of HEC-RAS was the desire to use the computational power of HEC-2 but present it in a very easy-to-use format. Several other improvements have been made. HEC-RAS is able to operate with mixed regime conditions, provided the proper boundary conditions are input into the program. HEC-RAS also includes the ability to model inline weirs and gates and multiple culvert openings, and has a new method for handling piers on bridges. The discussion of HEC-RAS that follows is designed to give the student a basic overview of the program and an appreciation for its strength as a hydraulic modeling tool. Its flexibility has been enhanced as its data are used in conjunction with other programs, as seen later in the chapter. For a more complete presentation of the program details, please refer to the HEC-RAS user's manual, which is downloadable, along with HEC-RAS itself, from the U.S. ACOE website at www.hec.usace.army.mil. In order to better introduce HEC-RAS, a model set up will be shown from a project on Big Creek, Texas, a creek that flows about 25 miles southwest of Houston.

Creating a Project

HEC-RAS divides the necessary input into two categories: geometric data and flow data. Both can be accessed through the Edit menu in the main program window or by clicking on one of the shortcut buttons in the same window. Doing so takes the user into either the Geometric Data Editor or the Steady Flow Editor. Each project has a main project file, which contains a list of all supporting files associated with that project, including geometry, flow, plan, and output files. A project can hold many different geometry and flow files and each combination of geometry and flow files that creates a plan file. Finally, the result of each run is then stored as an output file.

Geometric Data

The Geometric Data Editor window is where all physical and topographical data are input. The first step in creating a model is to click on a button on the left side of the window labeled **River Reach**. The user is then able to click the mouse in the viewing area and draw the river reach. The drawing in the window has no effect on the program computationally, so it does not need to be an accurate visual representation of what the reach looks like.

Once the reach is drawn, the cross sections can be entered. As cross sections are created, they are automatically placed on the drawing of the river reach. Clicking on the **Cross Section** button on the left of the Geometric Data Editor brings up the Cross Section Data window. To create a new cross section,

click on **Add a new Cross Section** in the Options menu. The program will prompt the user for the river station number. After the number is entered, one can input data for the cross section into the program. HEC-RAS requires a variety of data types. This window is where points along the cross section and their elevations are assigned. Distances to the next downstream cross section are needed along the left overbank (LOB), the channel, and the right overbank (ROB). Channel bank stations as well as contraction and expansion coefficients are also necessary. Note that the cross sections are shown from the perspective of the user looking downstream standing in the middle of the cross section.

The Manning n values can be entered in one of two different ways (see Table 7-1 for values). If there is no variation in the n value within a portion of the cross section, such as the ROB, then the n values can be directly entered into the existing fields. If there is variation in n values within a part of the cross section, choosing **Horizontal Variation in n Values** from the Options menu creates a new column next to the cross-sectional elevation field. Values in this column must be entered only when there is a change in n -values going from left to right across the cross section. Figure 7-16 shows an example of a completed Cross Section data input window.

Cross Section Data - low bridge

Exit Edit Options Plot Help

River: Big Creek Apply Data

Reach: Reach-1 River Sta.: 109047

Description

Del Row Ins Row

Cross Section X-Y Coordinates		
	Station	Elevation
1	9150	65
2	9575	60
3	9800	55
4	10010	54
5	10024	54
6	10050	41
7	10100	41
8	10110	41
9	10136	54
10	10140	54.5

Downstream Reach Lengths		
LOB	Channel	ROB
750	790	900

Manning's n Values		
LOB	Channel	ROB
0.1	0.04	0.1

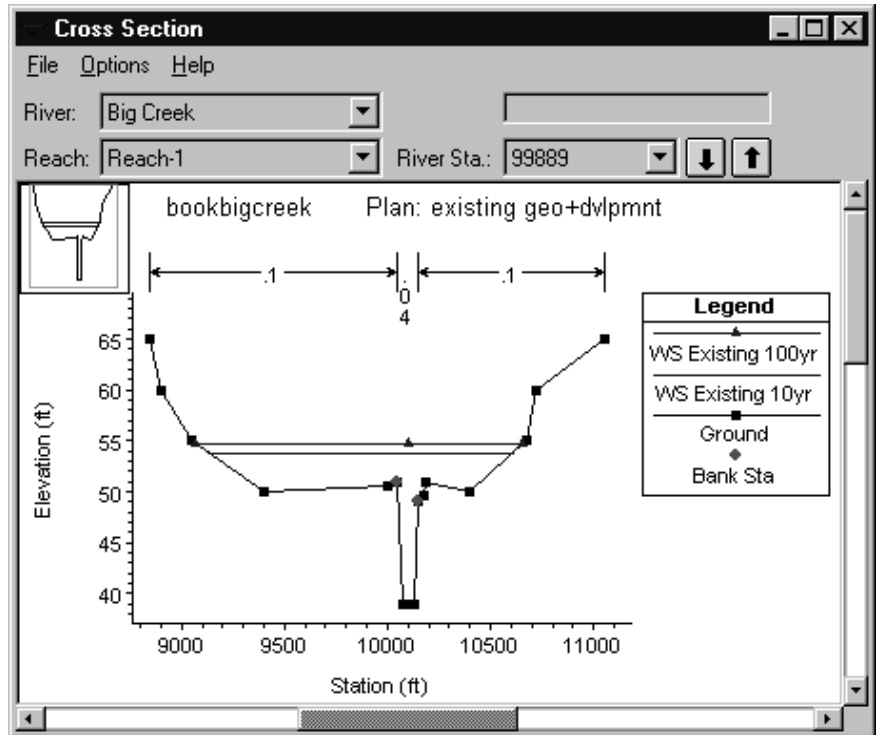
Main Channel Bank Stations	
Left Bank	Right Bank
10024	10136

Cont/Exp Coefficients	
Contraction	Expansion
0.1	0.3

Enter to move to next downstream river station location

Figure 7-16
Input window for
cross section.

Figure 7-17
Cross section with
water surface
profiles.



Other important options are accessed from the Cross Section Data window. Areas of ineffective flow as well as levees and blocked obstructions are defined from the Options menu. Areas of ineffective flow are areas in which flow is hampered. Examples of ineffective flow are often seen on the sides of the channel just next to bridges, where the water is rapidly contracting or expanding (see Fig. 7-15). Clicking on **Plot Cross Section** in the Plot menu brings up a plot of the cross section (Fig. 7-17). This is a good way to check if all the points are entered correctly and is another advantage of HEC-RAS compared to the original HEC-2. HEC-RAS cross-section data can be checked by simply clicking through the cross-section plots on the screen, using up and down arrows. The zoom feature is particularly useful to study specific details.

Bridges in HEC-RAS require four cross sections: two just a few feet away from each face of the bridge, one far enough upstream that flow has not yet begun to contract, and one far enough downstream that flow has completely expanded (Fig. 7-14). These cross sections are created just as any others, and clicking on the **Brdg/Culv** button in the Geometric Data Editor opens the Bridge Culvert Data window. From this window both bridges and culverts can be created. Click on Options and select **Add a Bridge and/or Culvert** to begin the process of creating the bridge. Defining a bridge's deck or roadway is very similar to defining the elevations and points along a cross section and can be done in the **Deck/Roadway** editor. The only difference is that two elevations must be

defined: the top chord and the low chord, which must be entered for both the upstream and downstream faces of the bridge (Fig. 7-13). These values are often the same for the two faces, so HEC-RAS provides a copy up-to-down button that simply copies the chord elevation data from one face to the other. A detailed analysis of the culvert program within HEC-RAS is contained in Section 9.3.

HEC-RAS allows the user to input individual bridge piers and to define the height and width of each pier, which in turn allows for pier widths that vary with elevation. Piers are defined in their own window, labeled **Pier** in the **Bridge Culvert Data** window. The **Pier** window is used for defining the size and location of the piers. A third window can be used to input sloping abutments. Finally, a fourth window allows the user to determine the method that HEC-RAS uses in calculating losses through the bridge. An option is even included for HEC-RAS to use the calculation method that results in the highest energy loss, which is a useful option that results in a more conservative model than if a single approach were used. Figure 7-18 shows a typical bridge section in HEC-RAS, and the zoom feature can be used to see more details.

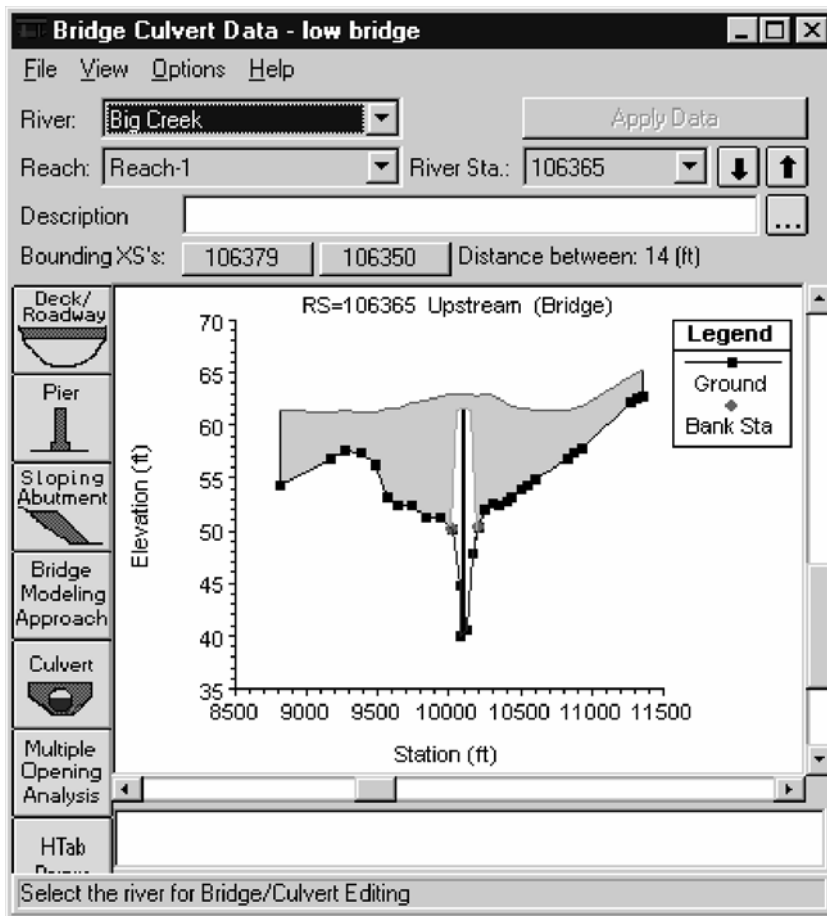


Figure 7-18
Bridge cross section.

Flow Data

The second major category of input data in HEC-RAS is flow data. HEC-RAS requires the user to select the reach and all the cross sections where a change in flow occurs. Once a flow is selected for a cross section, that flow is used on all cross sections upstream of the first until the program encounters a different flow. HEC-RAS also maintains the ability to model multiple profiles simultaneously. This allows the user to easily compare, for example, the 10-yr, 25-yr, 50-yr, and 100-yr floods on one graph. As mentioned before, the Steady Flow Data Editor is accessed by clicking **Edit** in the main program window, and then clicking **Steady Flow Data**. The flow data that are used are often generated by HEC-HMS and can be imported into HEC-RAS. This iteration between RAS and HMS can be used to compute and refine storage–outflow relationships that normally are used for flood routing in HMS (see Fig. 5–5).

The Steady Flow Data Editor has one other job of major importance, and that is setting the boundary conditions for the reach. A boundary condition must be selected for each profile, and one must keep in mind whether the flow is subcritical, supercritical, or mixed regime flow, as this determines whether to use downstream, upstream, or both boundary conditions. It should be noted that for floodplain analysis, downstream boundary conditions are generally used most often. Four types of boundary conditions can be used: known water surfaces, critical depth, normal depth, and given rating curves. Only one condition needs to be defined.

Running and Viewing Results

Once all geometric data, flow data, and boundary conditions have been entered, HEC-RAS can run the simulation. Click on **Run** and then **Steady Flow Analysis** in the main program window. This brings up a new window. If the user has several flow or geometry files, he/she can then choose which set of files to run. Each combination of geometry and flow files is stored as a separate plan, which can be given a unique name. Each time a new plan is computed, an output file is generated and saved with the results.

Results of HEC-RAS are useful in their ability to create various plots and tables of the output results. The displayed data are easily chosen from a menu so that the user is able to customize the data that are of interest. The cross-section output table lists a number of variables and associated values computed for each particular cross section. A typical cross-section output table is shown in Figure 7–19. All of the plots and tables that will be discussed in this section can be found by clicking on the **View** menu in the main program window.

A cross-section plot is one of the plots that are available (see Fig. 7–17 or 7–18). It plots any cross section in the reach and displays the water surfaces from as many or as few of the flow profiles as the user chooses.

Cross Section Output

File

Type

Options

Help

River:

Big Creek

Profile:

Existing 10yr

Reach

Reach-1

Riv Sta:

95305

Plan: existgeo+dvpr

Big Creek

Reach-1

RS: 95305

Profile: Existing 10yr

E.G. Elev (ft)	53.02	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.05	Wt. n-Val.	0.080	0.040	0.080
W.S. Elev (ft)	52.97	Reach Len. (ft)	2600.00	2034.00	1300.00
Crit W.S. (ft)		Flow Area (sq ft)	1184.01	1261.91	1250.53
E.G. Slope (ft/ft)	0.000112	Area (sq ft)	1184.01	1261.91	1250.53
Q Total (cfs)	3570.00	Flow (cfs)	457.92	2594.29	517.78
Top Width (ft)	937.95	Top Width (ft)	428.46	101.00	408.49
Vel Total (ft/s)	0.97	Avg. Vel. (ft/s)	0.39	2.06	0.41
Max Chl Dpth (ft)	14.47	Hydr. Depth (ft)	2.76	12.49	3.06
Conv. Total (cfs)	337600.6	Conv. (cfs)	43303.9	245332.1	48964.6
Length Wtd. (ft)	1988.80	Wetted Per. (ft)	428.50	105.40	408.57
Min Ch El (ft)	38.50	Shear (lb/sq ft)	0.02	0.08	0.02
Alpha	3.34	Stream Power (lb/ft s)	0.01	0.17	0.01
Frctn Loss (ft)	0.28	Cum Volume (acre-ft)	332.19	362.80	547.37
C & E Loss (ft)	0.00	Cum SA (acres)	154.50	31.18	256.49

Figure 7-19

Cross-section output table.

It can also display the energy grade lines and the critical depth, along with numerous other options. Other cross sections can be easily viewed by clicking on the arrow keys above the window. The water surface profile plot is another plot that displays the water surface and energy grade lines as a profile that runs lengthwise down the center of the channel (Fig. 7-20a). This plot can also graph the bank as well as the bridge stations on either side of the channel. This is a useful feature in that it shows not only where the water is overtopping the banks and bridges, but also by how much.

Another useful tool is the X-Y-Z perspective plot. Figure 7-20b shows the reach in three dimensions; the user may rotate the image about two axes. There is also a zoom-in command to allow the user to study a particular segment in more detail. This gives the user an easy way to see the relative extent that the floodwaters have traveled outside the bank and the relative slope change as one moves downstream along the profile. This plot can also display multiple profiles at once, making it easy to compare floods of different magnitude. This capability is a great improvement over HEC-2, allowing the user to visualize the floodplain in three-dimensional perspective.

Other options that exist in the View menu include the ability to create rating curves for any cross section, and tables that provide a vast array of calculated properties for each cross section and profile. A powerful feature

Figure 7-20a
Profile plot of reach.

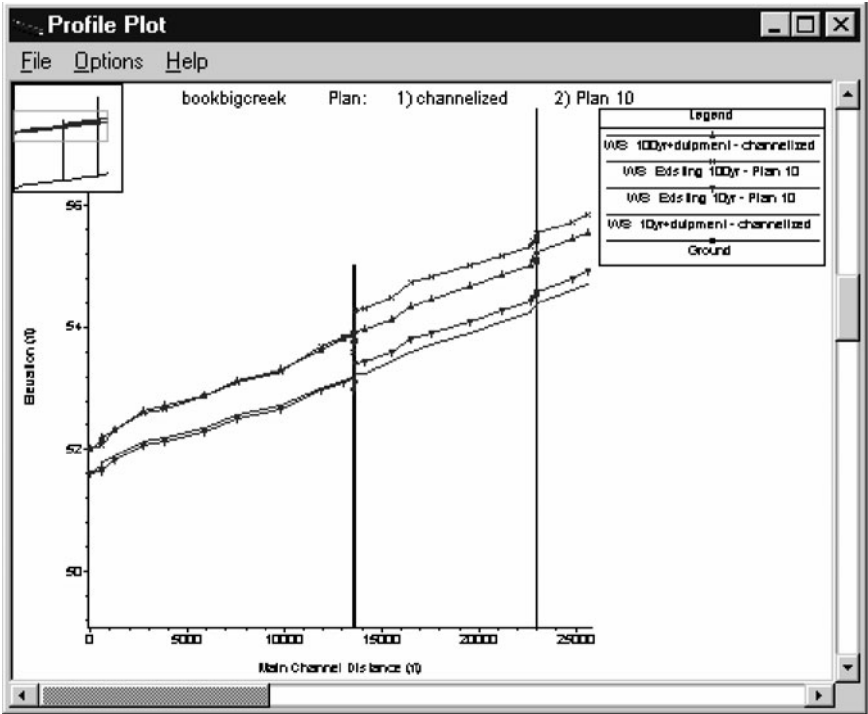
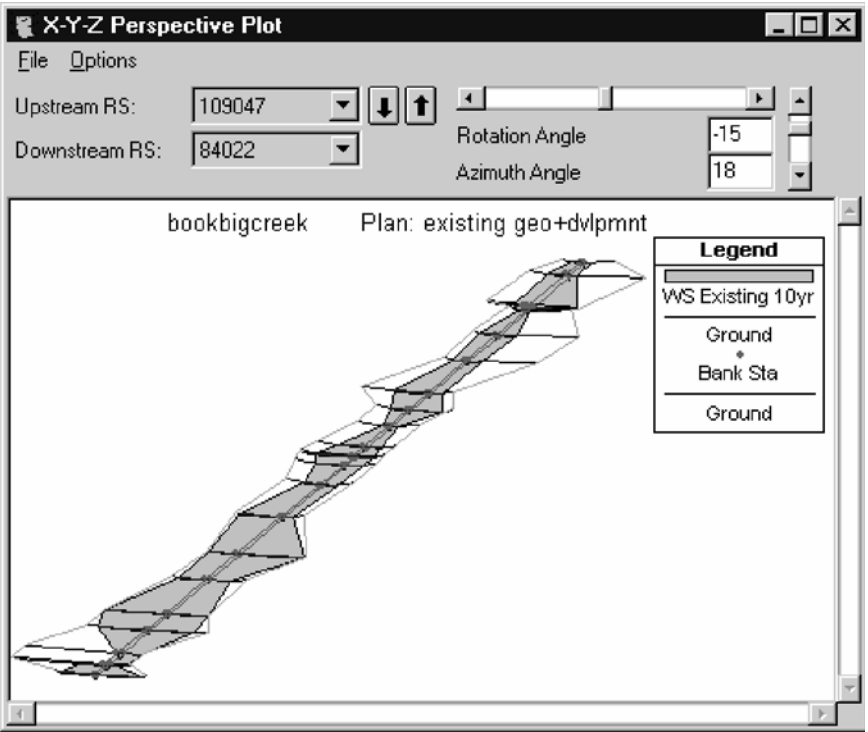


Figure 7-20b
3-D XYZ perspective plot of reach.



of all graphs and tables in HEC-RAS is the ability to customize the output. The user has great freedom in choosing which variables to display in a given form of output. Also, HEC-RAS, in the File menu of the main program window, can generate a report. Here as well, the user specifies what sort of information goes in the report. This is an important feature because other programs can use generated reports as input files, as described in a later section.

Channel Modification

Thus far, all the tools necessary to create and examine a stream or channel have been introduced. However, the job of the engineer is not always just to model the flooding, but to find ways to alleviate it. In a typical hydraulic analysis, this may involve making improvements to the channel in the form of deepening or widening a section. HEC-RAS has a window called **Channel Modification** that makes improving the channel a very simple process. The Channel Modification window can be found in the Tools menu in the Geometric Data Editor. The improved channel is referred to as the cut. Channel modification entails telling the model what range of cross sections will be widened and what the bottom width, side slope, and n values of the cut will become, along with a few other specifications.

The user begins by entering data into a part of the window entitled **Set Range of Values**. The range of cross sections to be modified is entered first, then the other properties of the cut. The Channel Modification window allows the invert elevations to be set in a number of ways. The elevations can remain the same, they can be set to a constant elevation, or a slope can be extended upstream from the downstream endpoint or downstream from the upstream endpoint. When the **Set Range of Values** portion of the window is completed, the **Apply Cuts to Selected Range** button is clicked. A table containing data from all the cross sections in the reach, not just the ones being modified, can be found at the bottom of the Channel Modification window. Once the cuts have been applied, the updated data appear in this table of cross sections. From here the user can make modifications to individual cross sections. Also, if the new channel does not run along the centerline of the existing channel, the center station must be entered in this table. Finally, the **Compute Cuts** button is pressed to update all the changes. A useful feature is the **Cut and Fill Areas** button. Pressing this button produces a report in tabular form that shows all of the calculated volumes of soil that must be excavated for the proposed channel, which is a good way to estimate the cost of the project. The modified channel can then be saved as a new geometry file and processed in the simulation.

Unsteady Flow Simulation

Version 3.0 of HEC-RAS allows, for the first time, for the unsteady flow analysis within a floodplain, and the user can actually view the change of

depth as a function of both space and time. The Unsteady NETwork simulator (UNET) was modified for HEC-RAS in order to simulate one-dimensional unsteady flow through a full network of open channels. In addition to solving the network system, UNET provides the user with the ability to apply several external and internal boundary conditions, including flow and stage hydrographs, rating curves, gated and uncontrolled spillways, pump stations, bridges, culverts, and levee systems. The UNET program was originally developed by Barkau (1992).

The program uses an unsteady flow solver at each node for each time step, and data are organized for each cross section. All HEC-RAS output options as well as complete animation of cross section, profile, and 3-D plots are available. This feature allows the user to view the dynamics of a flood wave as it moves downstream. As modern floodplain problems become more complex and the demand grows for more accurate solutions, the use of UNET within HEC-RAS will continue to increase. Other programs are also available to solve the dynamic equations presented in Chapter 4 and provide output as a flood wave moves downstream. A detailed unsteady-state example is provided in the textbook website hydrology.rice.edu.

EXAMPLE 7-7

APPLICATION OF HEC-RAS FOR BIG CREEK

Big Creek is a small creek located about 25 miles southwest of Houston, Texas. It is typical of many of the waterways in the Houston area in that it is characterized by a mild slope in the channel bed and a flat topography in the area surrounding. A portion of the creek has been modeled in HEC-RAS and will be used as the basis for the example. The project file is accessible to students in the textbook website listing in Appendix E. It contains a flow file and a geometry file of the existing conditions. The interested student should implement the channelization and bridge modifications for Big Creek by following along with directions in the book.

The problem is that a builder would like to develop a very large tract of land near the upstream portion of Big Creek, to consist of new houses and shopping centers. A hydrologic study is commissioned, and it is determined that the development would add 2200 cfs of additional flow to the 100-year storm, which has an existing flow rate of 4825 cfs at the mouth of the reach. This consequently raises the water surface elevation at the cross section farthest upstream to 57.54 ft, an increase of 1.7 ft above the existing elevation of 55.84 ft. Regulations require that a development cannot raise the water surface elevation downstream of the development by more than 0.01 ft. The developer has two options. The first is to build a detention pond in order to alleviate the impact of increased flow. However, this would tie up some of his land, which is at

a premium in this part of Houston. Therefore, the builder is interested in the feasibility of channelizing the portion of Big Creek downstream of the point of impact that has been modeled.

In addition to the increased flow rates, the HEC-RAS model shows that an existing bridge is causing a backwater effect in the reach of interest. The bridge is an old one and was poorly designed, such that it increases the water surface elevation by 0.37 ft just upstream of the bridge. In addition to the effect of channelization, the effect of raising the bridge should also be considered.

The model of Big Creek consists of 25 cross sections and two bridges over a reach length of 25,025 ft (4.74 mi). The current floodplain ranges in width from about 300 to about 3000 ft. The greatest changes in width due to increased flow in the creek occur in the upstream portion of the watershed due to geometry of the main channel. The depth of floodwaters outside the creek banks generally ranges from 1.0 to 5.0 ft.

In examining the geometry of the existing creek, one sees that the creek bed has a width of 60 ft. We enter the **Channel Modification** window. We begin by setting the range of cross sections to be channelized. We choose to begin at Section 109047 and end at Section 84629, a distance of 24,418 ft. Continuing in the **Select Range of Values** section of the window, we select center cuts and a width of 80 ft. The invert elevation is left blank, meaning that the existing depth of the channel will remain the same. A left and right side slope of 3 is chosen, meaning that the side slope is equal to 3 ft horizontally to every 1 ft vertically. The n -value for the cut is entered to be .03, which is a typical value used for channelized reaches. Once these values are entered, the **Apply Cuts to Selected Range** button is pushed. If there are no other modifications that need to be made to the cut, then the **Compute Cuts** button can be pushed. As mentioned before, a useful feature of the Channel Modification program is the **Cut and Fill Areas** button. This calculates and reports the total volume of soil that must be excavated. We see that it is necessary to remove 360,988 yd³ for this project. Given a base estimate of \$4.00/yd³, it is estimated that this project will cost \$1.44 million.

Finally, the **Create Modified Geometry** button is pushed, and a new window is opened. The new geometry is saved as a new geometry file with a name such as "80ft channel." Finally, we return to the HEC-RAS main window and select **Run**, then **Steady Flow Analysis**. From the dropdown menu, we choose "80ft channel" as our geometry file, 100 yr dev as our flow file, and then run the program. We can view the profile output table and see that the upstream water surface elevation has been lowered to 55.61 ft, which is almost exactly what it was before the proposed development. Therefore, this channelization project was successful in reaching its goal of properly lowering the water surface elevation.

It is now the responsibility of the developer to weigh the advantages of the channelization against the economic and environmental costs of the project. Channelization often leads to destruction of the riparian habitat, and this must be seriously considered. The developer may also look into the option of using an off-line detention pond as a means of flood attenuation, which may not create the environmental damage that channelization may incur. The option of using a detention pond is explored using HEC-HMS, which is discussed in Chapter 5.

The issue of raising the bridge needs to be examined. The bridge in question is located at Section 97030. The area underneath the bridge that is open to flow is too small for the flow at the rate that it must pass through, and the physical mass of the bridge structure is creating a damming effect on the water. To examine this backwater effect, open a water surface profile plot from the **View** menu in the main project window. The sudden increase seen in the water surface elevation upstream of bridge Section 97030 indicates that the bridge is holding water back. One solution to this problem is to create a greater area for flow by raising the old bridge. In HEC-RAS, the user enters the Deck/Roadway window from the Bridge Culvert Data Editor. The high and low chords can be altered so that over the channel section, the high and low chords are raised to 57 and 55 ft, respectively, and pier heights should also be modified, using Pier Editor. The changes are saved and the results are run in HEC-RAS.

The final results are easily compared using the water surface profile plot. The newest geometry, with the improved channel and raised bridge, can be compared against the original conditions. It is seen that downstream of bridge Section 97030, the two water surface profiles match up almost exactly. Upstream of bridge Section 97030, the original conditions actually have a slightly higher water surface elevation due to the backwater effect at the bridge. It is seen then that the channelization has maintained the water surface elevation, and the raised bridge actually improves the water surface elevation.

Table 7-3 depicts a flowchart of steps to be implemented in running the HEC-RAS model for a new project, such as Big Creek described in this example. In fact, all of the figures referred to in the table are from the Big Creek application and show a typical series of data and plots that one would generate as part of a floodplain analysis with the model. The RAS figures are described in more detail throughout this section. The interested student is encouraged to download HEC-RAS from the HEC ACOE website and to utilize the Big Creek data set, which is available at the textbook site for this textbook (see Appendix E). A few of the homework problems (7.26 to 7.30) help guide the first-time user through the application of the model.

Table 7-3. Basic Guide to Creating a HEC-RAS Model

Step Number	Description of HEC-RAS Operation
1	Open HEC-RAS, start a ' New Project ' under the ' File ' menu, name the project.
2	Enter Geometric Data Editor and push ' River Reach '. Draw the reach, name it, and save the geometry file.
3	Enter Cross Section Editor. Create cross sections by providing all necessary data on X-Y coordinates, downstream reach lengths, Manning's n -values, bank stations, and contraction/expansion coefficients. See Figure 7-16. The cross section can be examined as a plot, as seen in Figure 7-17.
4	Enter Bridge/Culvert Data Editor. Enter data for Deck/Roadway, Pier, and Bridge Modeling Approach. If it is a culvert, enter the data specific to the culvert in the Culvert Data Editor. A plot of the bridge or culvert can be created, as seen in Figure 7-18.
5	Save geometry file (<i>projectname.g01</i>).
6	Return to main project window and enter Steady Flow Editor.
7	Enter number of flow profiles and enter reach boundary conditions by selecting either upstream or downstream boundary conditions and then choosing to define a known water surface, critical depth, normal depth, or a rating curve.
8	Add flow change locations and enter the flowrate at each.
9	Save the flow data (<i>projectname.f01</i>).
10	From main project window, select ' Run ' and then ' Steady Flow Analysis ... '
11	Check to make sure proper geometry and flow files are selected before pressing ' Compute '.
12	View results by choosing one of the output options in the ' View ' menu in the main project window. Some of the more popular forms of output are: — Cross-section plot (Fig. 7-17) — Detailed output table (Fig. 7-19) — Water surface profile plot (Fig. 7-20a) — X-Y-Z perspective plot (Fig. 7-20b)

This chapter presents a review of uniform and nonuniform open-channel flow. Uniform flow computations with Manning's equation are derived and examples are presented for various channel shapes. Critical flow conditions are defined in which the Froude number is used to characterize channel conditions as subcritical or supercritical. The gradually varied flow equations are derived based on energy loss through a channel. These equations are used to classify water surface profiles for backwater computations, which are presented in the examples.

The HEC_RAS model is an extremely flexible tool that can be used to calculate water surface profiles for natural or man-made streams. It contains all of the necessary hydraulic coefficients and equations for most flow conditions that would be encountered and can handle a number of options for bridges, multiple profiles, ineffective

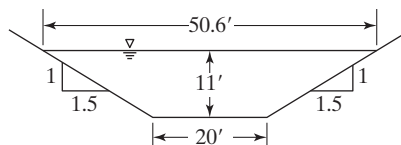
SUMMARY

areas, encroachments, and split flow. The detailed example presented in the chapter is designed to represent typical input and output, and more detailed applications require familiarity with the HEC-RAS user's manual. HEC-RAS represents a marked improvement over the original HEC-2 due to its expanded user interface and its improved output capabilities. See Haestad et al. (2003) and the HEC-RAS user's manual (Hydrologic Engineering Center, 1995, 2002).

PROBLEMS

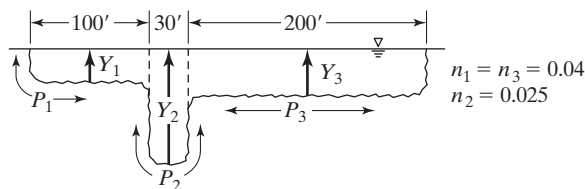
- 7.1. The Colorado River System Aqueduct has the cross section shown in Figure P7-1. When the water in the aqueduct is 11 ft deep, flow is measured as 1600 cfs. If $n = 0.014$, what is S_0 in (a) ft/ft and (b) ft/mi?

Figure P7-1



- 7.2. A rectangular open channel that is 2 m wide has water flowing at a depth of 0.50 m. Using $n = 0.014$, find the rate of flow in the channel if S_0 is (a) 0.002 m/m, (b) 0.006 m/m, and (c) 0.012 m/m.
- 7.3. A channel has the irregular shape shown in Figure P7-3, with a bottom slope of 0.0016 ft/ft. The indicated Manning's n -values apply to the corresponding areas only. Assuming that $Q = Q_1 + Q_2 + Q_3$, find the rate of flow in the channel if $y_1 = 2.5$ ft, $y_2 = 10$ ft, and $y_3 = 3$ ft.

Figure P7-3



- 7.4. Water is flowing 2.5 m deep in a rectangular channel that is 2.5 m wide. The average velocity is 5.8 m/s and $C = 100$. What is the slope of the channel? (Use Chezy's formula.)
- 7.5. A triangular channel with side slopes at 45 degrees to the horizontal has water flowing through it at a velocity of 10 ft/s. Find Chezy's roughness coefficient C if the bed slope is 0.05 ft/ft and the depth is 4 ft.
- 7.6. Water is flowing at a rate of 900 cfs in a trapezoidal open channel. Given that $S_0 = 0.001$, $n = 0.015$, bottom width $b = 25$ ft, and the side slopes are 1:1.5, what is the normal depth y_n ?

- 7.7. Find the normal depth y_n for the triangular channel shown in Figure P7-7 if $S_0 = 0.0003$ m/m, $Q = 50$ m³/s, and $n = 0.030$.

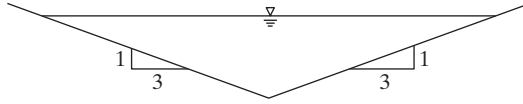


Figure P7-7

- 7.8. Determine the critical depth and the critical velocity for the Colorado River System Aqueduct (Problem 7.1) if $Q = 1500$ cfs.
- 7.9. Find the critical depth and critical velocity for the triangular channel of Problem 7.7 if Q is (a) 20 m³/s and (b) 50 cfs.
- 7.10. Determine the local change in water surface elevation caused by a 0.2-ft-high obstruction in the bottom of a 10-ft-wide rectangular channel on a slope of 0.0005 ft/ft. The rate of flow is 25 cfs and the unobstructed flow depth is 0.9 ft. (See Fig. P7-10). Assume no head loss.

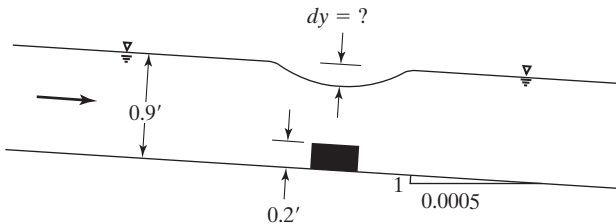
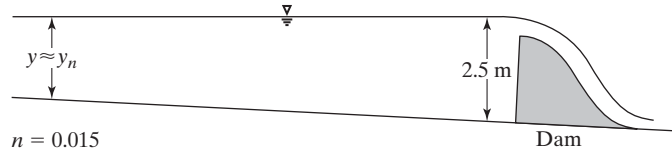


Figure P7-10

- 7.11. A rectangular channel with $n = 0.012$ is 5 ft wide and is built on a slope of 0.0006 ft/ft. At point a , the flow rate is 60 cfs and $y_a = 3$ ft. Using one reach, find the distance to point b where $y_b = 2.5$ ft and determine whether this point is upstream or downstream of point a .
- 7.12. If a channel with the same cross-sectional and flow properties as the channel of Problem 7.11 is laid on a slope of 0.01 ft/ft, determine whether the flow is supercritical or subcritical. Find the depth of flow at a point 1000 ft downstream from the point where $y = 1.5$ ft. (A trial-and-error solution may be necessary.)
- 7.13. Classify the water surface profiles according to Table 7-2 of (a) Problem 7.11 and (b) Problem 7.12.
- 7.14. Classify the bed slopes (mild, critical, steep) of the channels of the following problems: (a) Problem 7.1, (b) Problem 7.6, and (c) Problem 7.7.
- 7.15. A stream bed has a rectangular cross section 5 m wide and a slope of 0.0002 m/m. The rate of flow in the stream is 8.75 m³/s. A dam is built across the stream, causing the water surface to rise to 2.5 m just upstream of the dam. (See Fig. P7-15.) Using the step method illustrated in Example 7-5, determine the water surface profile upstream of the dam to a point where $y = y_n \pm 0.1$ m. Assume $n = 0.015$. How far upstream does this point occur?

Figure P7-15



- 7.16.** A rectangular concrete channel ($n = 0.020$) changes from a mild slope to a steep slope. The channel is 20 m wide throughout, and the rate of flow is $180 \text{ m}^3/\text{s}$. If the slope of the mild portion of the channel is 0.0006 m/m , determine the distance upstream from the slope change to the point where $y = 3.0 \text{ m}$. (*Hint: Use the same step method as in Example 7-5, with y intervals of 0.1 m .)*
- 7.17.** A rectangular channel 1.4 m wide on a slope of 0.0026 m/m has water flowing through it at a rate of $1 \text{ m}^3/\text{s}$ and a depth of 0.6 m . A cross section of the channel is constricted to a width of 0.9 m . What is the change in water surface elevation at this point?

Problems 7.18 through 7.20 refer to the watershed shown in Figs. P7-18(a) and (b).

Cypress Creek has a rectangular cross section with a bottom width of 200 ft , $n = 0.03$, and $S_0 = 0.001 \text{ ft/ft}$. East Creek has the same characteristics except that the bottom width is 100 ft . The 100-yr storm hydrographs are shown for both creeks. The assumption is made that the flow in the creeks remains constant above point C and that the 100-yr hydrograph for Cypress Creek includes the inflow from East Creek at point C .

- 7.18.** Determine the normal and critical depths for both creeks for the 100-yr peak flow.

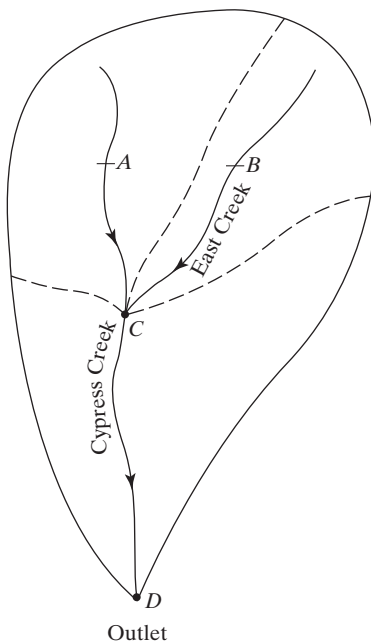


Figure P7-18(a)

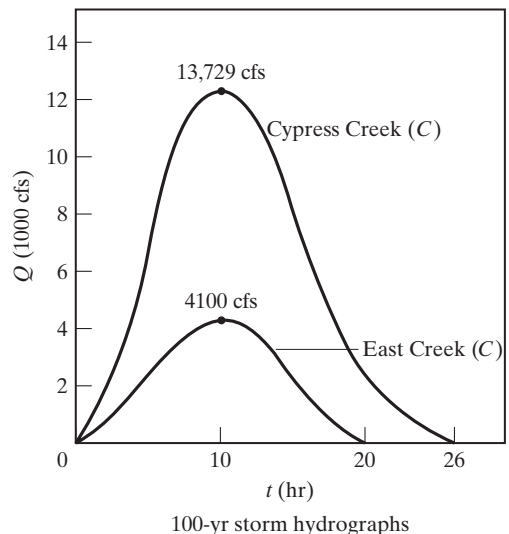


Figure P7-18(b)

- 7.19.** Assume that East Creek meets Cypress Creek as shown. Using a starting elevation at point C consistent with the 100-yr flow in Cypress Creek, develop the 100-yr water surface profile for East Creek. Use six points between the starting elevation and the elevation $y = 1.1y_n$.
- 7.20.** A developer proposes improvements to the East Creek subwatershed that will increase the peak flow of the 100-yr storm by 1000 cfs. The developer contends that there will be no change in the 100-yr elevations on the East Creek above point C. Determine the 100-yr water surface profile for East Creek. Use an interval of $\Delta y = 0.15$ ft. up to $y = 1.1y_n$. Discuss.
- 7.21.** Lost Creek has a rectangular channel 2 mi in length (10,560 ft) with a wooden bridge in the middle of this reach ($x = 5280$ ft). (See Fig. P7–21.) The channel is dredged earth ($n = 0.025$) with a bottom width of 200 ft and a bed slope of 0.001. The computed 100-yr peak flow is 12,000 cfs for the entire 2-mi reach.
- Compute the normal depth.
 - Compute the critical depth.

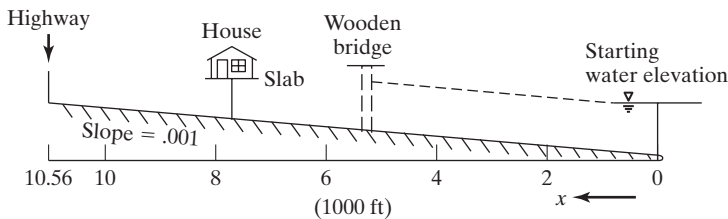


Figure P7–21

- 7.22.** For Lost Creek in Problem 7.21, the initial downstream water elevation is 10 ft. A house is to be built at a distance of $x = 2640$ ft upstream of the bridge.
- At what elevation should the house foundation be built to ensure 100-yr flood protection (neglect effects of the bridge)? Note the water velocity just downstream of the bridge.
 - Compute the head loss through the bridge according to the Yarnell equation [Eq. (7–37)] using $K = 0.95$, $a = 1/10$, and the velocity just downstream of the bridge as computed in part (a).
 - Determine slab elevation of the house for flood safety, taking into account the effects of the bridge. Compare the slab elevation in part (a) to that computed in part (b). What effect (if any) does the bridge have on the slab elevation?
- 7.23.** Derive the backwater curve for Example 7–5 with a starting downstream elevation of 8.0 ft. Repeat the calculation for 9.0 ft. All other parameters remain the same.
- 7.24.** Set up the input data structure to run Example 7–5 using HEC-RAS, with a starting downstream elevation of (a) 8.0 and (b) 9.0 ft.
- 7.25.** Refer to the HEC-RAS user's manual as needed. Run the sample river dataset provided in the tutorial.

For Problems 7.26–7.30, set up the Big Creek data for Example 7–7, available from the textbook website (see Appendix E).

- 7.26.** Run the existing condition 100-year floodplain and plot the profile output with HEC-RAS. Rerun the model with a 25% increase in flow rate and compare.
- 7.27.** Evaluate the effect of removing the upstream bridge at Section 106365 on the backwater profile in HEC-RAS.
- 7.28.** Evaluate the effect of increasing the downstream boundary condition water elevation by 2.0 ft in HEC-RAS.
- 7.29.** Run the existing condition 100-year floodplain and plot three cross sections as well as the X-Y-Z perspective plot with HEC-RAS.
- 7.30.** Set up the Big Creek data for Example 7–7, available from the Prentice Hall website (see Appendix E). Investigate the effects of changing Manning's n -values for the channel from 0.04 to 0.06 and from 0.08 to 0.10 for ROB and LOB areas in HEC-RAS.

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Chapter 8

Ground Water Hydrology



Groundwater well drilling.

Our study of hydrology up to this point has concentrated on various aspects of surface water processes, but an engineering hydrologist also must be able to address issues in ground water hydrology and well mechanics. This chapter presents a concise treatment of ground water topics, including properties of ground water aquifers, ground water flow, governing equations, well hydraulics, and ground water modeling techniques. This represents a minimum coverage for the practicing hydrologist or engineering student. Several excellent textbooks in the ground water area offer more detailed treatment, including discussion of important contamination issues (Bear, 1979; Todd and Mays, 2005; Fetter, 1994; Bedient et al., 1999; Charbeneau, 2000).

Ground water hydrology is of great importance because of the use of **aquifer systems** for water supply and because of the threat of contamination from waste sites at or below the ground surface. Recent attention has focused on ground water contamination problems associated with industrial or chemical spills and leaks. Properties of the **porous media** and subsurface geology govern both the rate and direction of ground water flow in any

8.1 INTRODUCTION

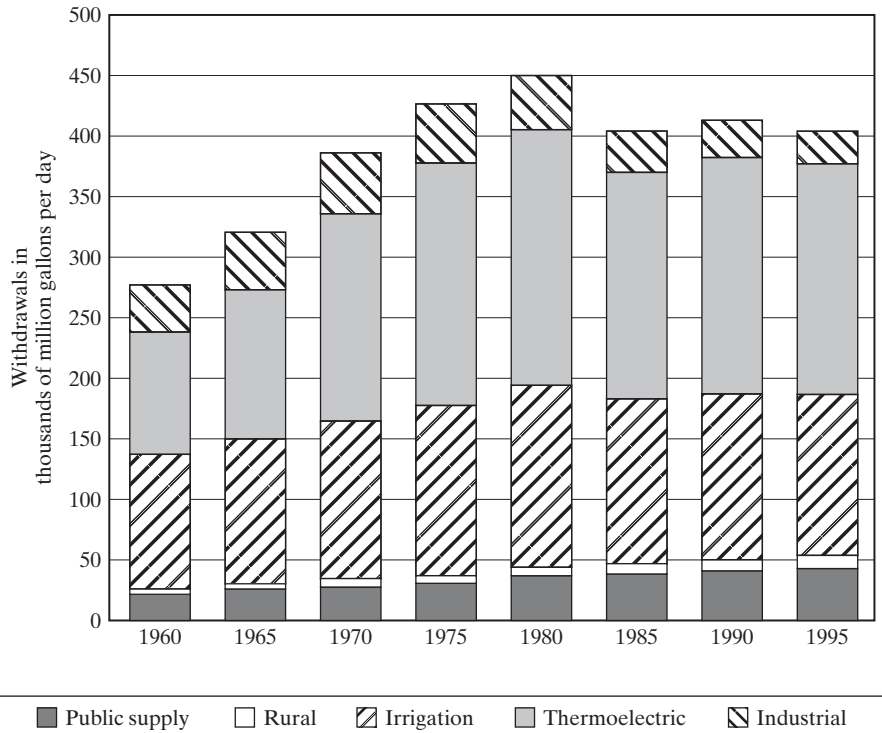


Figure 8–1

Trends in ground water use in the United States, 1960–1995. (Source: Based on data provided by Solley, 1998, U.S. Geological Survey Circular 1.)

aquifer system. The injection or accidental spill of waste into an aquifer or the pumping of the aquifer for water supply may alter the natural hydraulic flow patterns. The hydrologist must have a working knowledge of methods that have been developed to predict rates of flow and directions of movement in ground water systems. Only then can one possibly address problems of ground water contamination.

Ground water is an important source of water supply for municipalities, agriculture, and industry. Figure 8–1 indicates the distribution (1965 to 1995) of various types of ground water use in the United States, and it can be seen that irrigation and thermoelectric account for the greatest percentage. Western and midwestern areas of the United States are generally much more dependent on ground water than the eastern areas, except for Florida and Mississippi, which depend on ground water to a large extent. Techniques for the design of water supply systems that rely in part on ground water aquifers are an important part of engineering hydrology.

The study of ground water became a very big issue in the late 1970s and early 1980s with the discovery of numerous hazardous and leaking

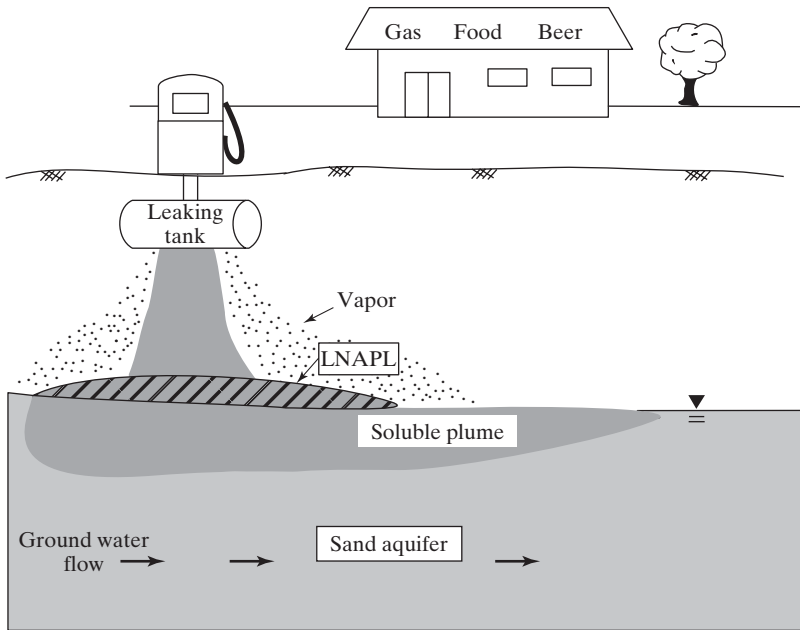


Figure 8-2
Typical hydrocarbon spill.

waste sites across the United States. Ground water became a common household term when over 1500 sites were placed on the National Priority List from the U.S. EPA. But thousands of other sites were also evaluated, monitored, sampled, and remediated over the past two decades. One of the most widely known contamination problems was associated with leaking underground storage tanks. The fuels that leaked caused enormous problems in shallow aquifer systems across the country and continue to leak in a number of communities despite massive cleanup costs. The engineering hydrologist today must be able to deal with mechanisms of ground water flow and contamination, as shown in Figure 8-2. More details on ground water transport can be found in Charbeneau (2000) and Bedient et al. (1999).

The U.S. Geological Survey (USGS) has primary responsibility for the collection of ground water data and evaluation of these data in terms of impacts on water supply, water quality, water depletion, and potential contamination. Results published by the USGS provide information on ground water levels and water quality data throughout the United States. Other primary sources of information are state water resources agencies, the American Geophysical Union, and the National Water Well Association. Journals such as *Ground Water*, *Water Resources Research*, and *ASCE* journals are some of the major mechanisms for exchange of technical information.

8.2 Vertical Distribution of Ground Water

PROPERTIES OF GROUND WATER

Ground water can be characterized according to vertical distribution, as shown in Figure 8–3, which indicates the main divisions of subsurface water. The unsaturated zone or **vadose zone** extends above the water table and includes the soil water zone down to the water table, which divides the unsaturated zone from the saturated zone. The **water table** is defined as the level to which water will rise in a well drilled into the saturated zone. Thickness may vary from a few feet for high water table conditions to several hundred feet in arid regions of the country, such as Arizona or New Mexico.

The **soil water zone**, which extends from the ground surface down through the major root zone, varies with soil type and vegetation. The amount of water present in the soil water zone depends primarily on recent exposure to rainfall and infiltration. Following rainfall, the water content in this zone will decrease due to downward drainage as well as losses to evaporation and root uptake. **Hygroscopic water** remains adsorbed to the surface of soil grains, while **gravitational water** drains through the soil under the influence of gravity.

Capillary water is held in the zone just above the water table and exists because water can be pulled upward from the water table by surface tension. The capillary zone, or fringe, extends from the water table up to the limit of **capillary rise**, which varies inversely with the pore size of the soil and directly with the surface tension. Capillary rise can range from a few cm for fine gravel to more than 200 cm for silt (Todd and Mays, 2005). Just above the water table almost all pores contain capillary water, but then the water content

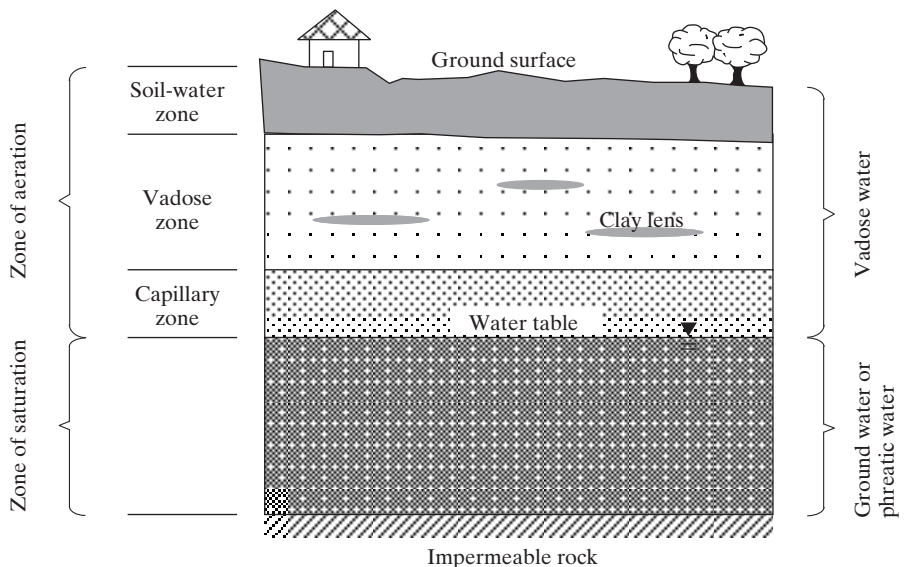


Figure 8–3
Vertical zones of subsurface water.

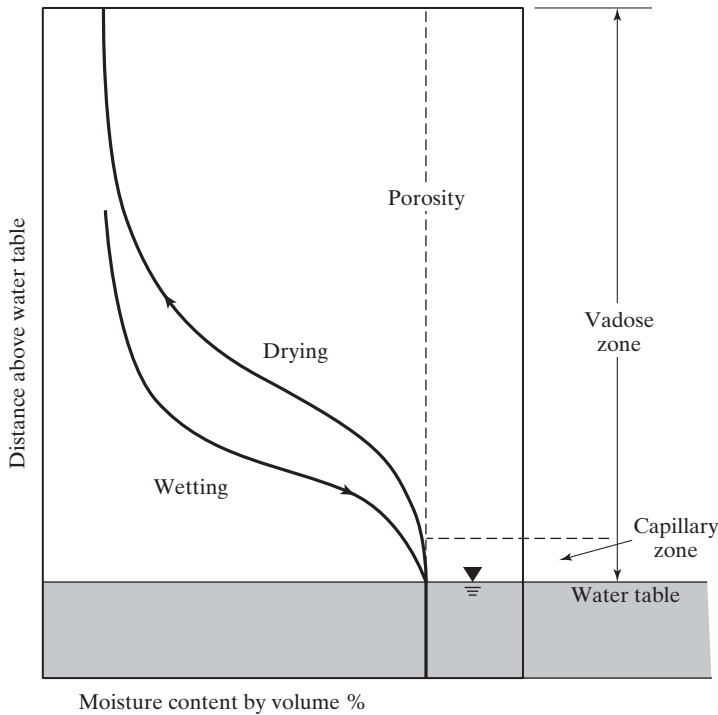


Figure 8-4
Typical soil moisture relationship.

decreases quickly with height, depending on the type of soil. A typical soil moisture curve is shown in Figure 8-4.

In the saturated zone, which occurs beneath the water table, the **porosity** is a direct measure of the water contained per unit volume, expressed as the ratio of the volume of voids to the total volume. Porosity ranges from 25% to 35% for most aquifer systems. Only a portion of the water can be removed from the saturated zone by drainage or by pumping from a well. **Specific yield** is defined as the volume of water released from an unconfined aquifer per unit surface area per unit head decline in the water table. Fine-grained materials yield little water, whereas coarse-grained materials provide significant water and thus serve as aquifers. In general, specific yields for unconsolidated formations fall in the range of 7% to 25%.

For water supply or to study the characteristics of aquifer systems, wells are often installed. A **well** is a vertical hole dug into the earth, usually cased with metal or PVC plastic up to the land surface. Often, the portion of the well hole that is open to the aquifer is **screened** to prevent aquifer material from entering the well. A vertical penetration used to collect soil or water samples that is not cased is called a borehole. Wells are generally placed into the saturated zone and are pumped for water supply for municipal, agricultural, and industrial customers. Wells can also be used for recharge, disposal of contaminated water or saltwater, and for water level observation. For more detail on well construction methods, see Section 8.11.

Aquifer Systems

An **aquifer** can be defined as a formation that contains sufficient permeable material to yield significant quantities of water to wells and springs. Aquifers are generally areally extensive and may be overlain or underlain by confining beds. Many aquifers in the United States have been extensively evaluated for water supply and for contamination problems. Several aquifers of note include the Edwards limestone near Austin, Texas, Ogallala in West Texas, and the Floridan Aquifer, the subject of a detailed modeling case study in Section 8.12.

An **aquiclude** is saturated and is a relatively impermeable confining unit, such as clay, that might act as a confining layer above or below an aquifer system. An **aquitard** is a saturated, low permeability stratum, such as a silty clay, that may leak water from one aquifer to another.

Aquifers can be characterized by the porosity of the rock or soil, expressed as the ratio of the volume of voids V_v to the total volume V . Porosity may also be expressed by

$$n = \frac{V_v}{V} = 1 - \frac{\rho_b}{\rho_m}, \quad (8-1)$$

where ρ_m is the **density** of the grains and ρ_b is the **bulk density**, defined as the oven-dried mass of the sample divided by its original volume. Table 8-1 shows a range of porosities for a number of aquifer materials, but usually 25% to 30% is assumed for most aquifers. Fractured rock or limestone can have lower porosities in the range of 1% to 10%.

Unconsolidated geologic materials are normally classified according to their size and distribution. Soil classification based on particle size is shown in Table 8-2. Particle sizes are measured by mechanically sieving grain sizes larger than 0.05 mm and measuring rates of settlement for smaller particles in suspension. A typical particle-size distribution graph is shown in Figure 8-5. The **uniformity coefficient**, defined as D_{60}/D_{10} , indicates the relative uniformity of the material. A uniform material such as fine beach sand has a low uniformity coefficient, while a well-graded material such as **alluvium** has a high coefficient (Fig. 8-5).

The texture of a soil is defined by the relative proportions of sand, silt, and clay present in the particle-size analysis and can be expressed most easily on a triangle diagram of soil textures. These are presented in any standard

Table 8-1. Representative Ranges of Porosity

Material	Porosity (%)
Sand or gravel, well sorted	25 to 50
Sand and gravel, mixed	20 to 35
Glacial till	10 to 20
Silt	35 to 50
Clay	33 to 60

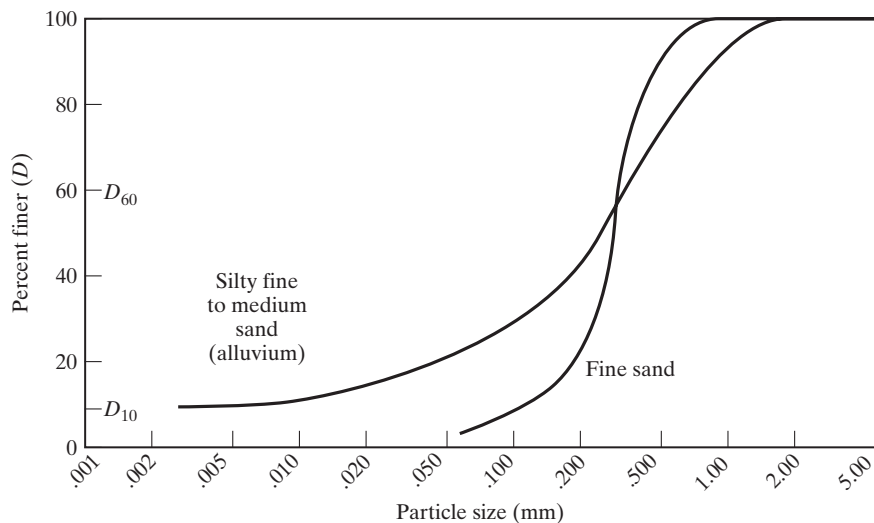
Table 8-2. Soil Classification Based on Particle Size*

Material	Particle Size (mm)
Clay	<0.004
Silt	0.004–0.062
Very fine sand	0.062–0.125
Fine sand	0.125–0.25
Medium sand	0.25–0.5
Coarse sand	0.5–1.0
Very coarse sand	1.0–2.0
Very fine gravel	2.0–4.0
Fine gravel	4.0–8.0
Medium gravel	8.0–16.0
Coarse gravel	16.0–32.0
Very coarse gravel	32.0–64.0

*After Morris and Johnson, 1967.

soils textbook. For example, a soil with 30% clay, 60% silt, and 10% sand is referred to as a silty clay loam.

Most aquifers can be considered underground storage reservoirs that receive recharge from rainfall or from an artificial source. Water flows out of an aquifer due to gravity or to pumping from wells. Aquifers may be classified as **unconfined**, depending on the existence of a water table, defined by levels in shallow wells. A **confined** aquifer is one that is overlain by a confining unit and is under pressure. A **leaky** confined aquifer represents a stratum that allows water to flow from above through the confining zone. A **perched water table** is an example where an unconfined water body sits on top of a clay lens, separated from the main aquifer below.

**Figure 8-5**
Particle-size
distribution graph
for two geologic
samples.

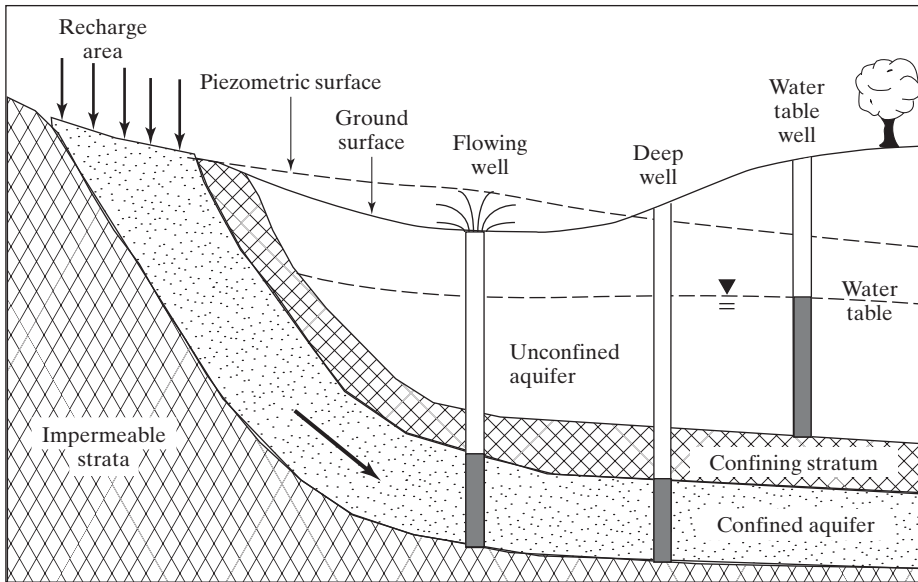


Figure 8-6
Schematic cross section illustrating unconfined and confined aquifers.

Figure 8-6 shows a vertical cross section illustrating unconfined and confined aquifers. An unconfined aquifer is one in which a water table exists, which often rises and falls with changes in rainfall and recharge. Shallow wells are usually placed to help define the level of water and the general direction of flow, based on the slope of the water table. Confined aquifers occur where ground water is confined by a relatively impermeable stratum, or confining unit, and water is under pressure greater than atmospheric. If a well penetrates such an aquifer, the water level will rise above the bottom of the confining unit. If the water level rises above the land surface, a flowing well or spring results and is referred to as an **artesian well**. Details on wells are provided in Section 8.11.

A recharge area supplies water to a confined aquifer, and such an aquifer can convey water from the recharge area to locations of natural or artificial discharge. The **piezometric surface** (potentiometric surface) of a confined aquifer is the hydrostatic pressure level of water in the aquifer, defined by the water level that occurs in a lined penetrating well. It should be noted that a confined aquifer can become unconfined when the surface falls below the bottom of the upper confining bed. Contour maps and profiles can be prepared of the water table for an unconfined aquifer or the piezometric surface for a confined aquifer. These **equipotential lines** are lines of constant head and will be described in more detail in Section 8.7. Figure 8-7 depicts a simple flow net, which is a set of constant head and orthogonal streamlines to indicate the direction of flow (from higher head to lower head) in a ground water system.

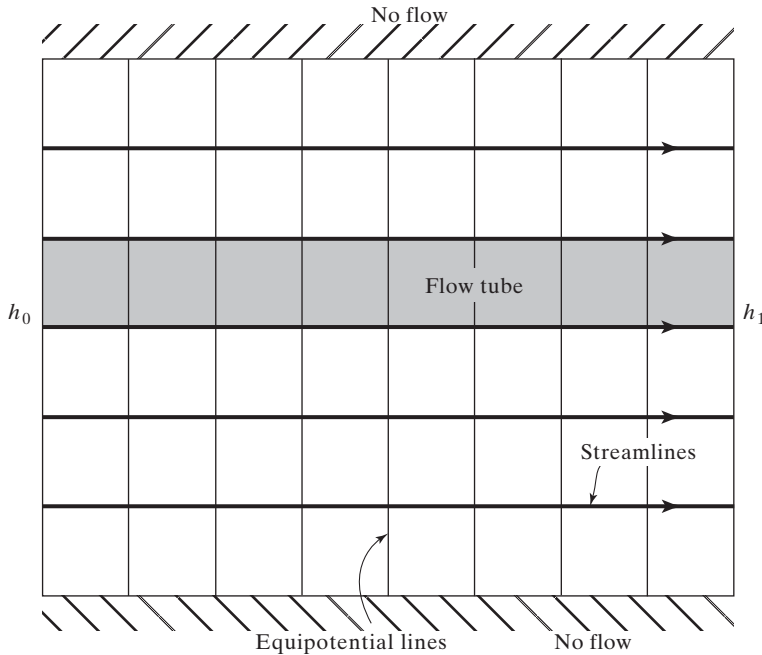


Figure 8-7
Simple flow net.

A parameter of some importance is the **storage coefficient** S , which relates to the water-yielding capacity of an aquifer. S is defined as the volume of water that an aquifer releases from or takes into storage per unit surface area per unit change in piezometric head. For a confined aquifer, values of S fall in the range of 0.00005 to 0.005, indicating that large pressure changes produce small changes in the storage volume. For unconfined aquifers, a change in storage volume is expressed simply by the product of the volume of aquifer lying below the water table at the beginning and end of a period of time and the average specific yield of the formation. Thus, the storage coefficient for an unconfined aquifer is approximately equal to the specific yield, or percentage of aquifer made up of water (typically 7% to 25%). Storage coefficient is described in more detail in Section 8.5.

Darcy's Law

The movement of ground water is well established by hydraulic principles reported in 1856 by Henri Darcy, who investigated the flow of water through beds of permeable sand. Darcy discovered one of the most important laws in hydrology—that the flow rate through porous media is proportional to the head loss and inversely proportional to the length of the flow path. Darcy's law serves as the basis for present-day knowledge of ground water flow and well hydraulics and forms the basis for the governing ground water flow equations (Section 8.5).

8.3 GROUND WATER MOVEMENT

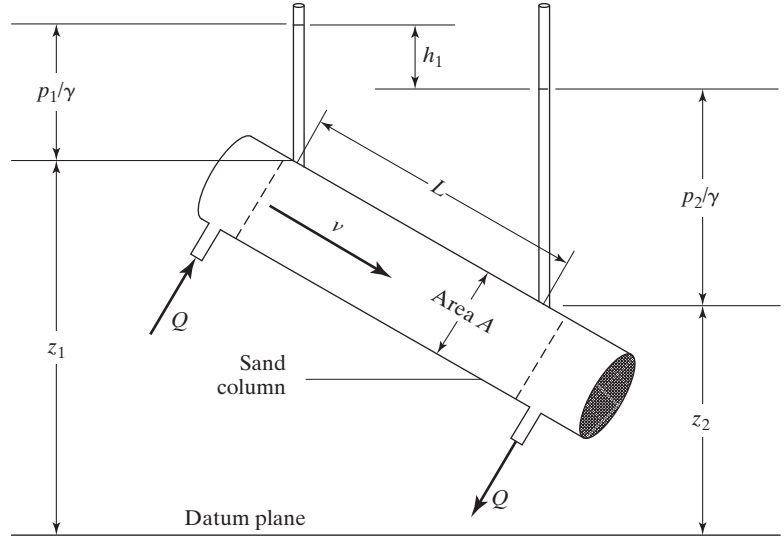


Figure 8–8
Head loss through a sand column.

Figure 8–8 depicts the experimental setup for determining head loss through a sand column, with **piezometers** located a distance L apart. Total energy for this system can be expressed by the Bernoulli equation

$$\frac{p_1}{\gamma} + \frac{v_1^2}{2g} + z_1 = \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + z_2 + h_1, \quad (8-2)$$

where

- p = pressure,
- γ = specific weight of water.
- v = velocity,
- z = elevation,
- h_1 = head loss.

Because velocities are very small in porous media, velocity heads may be neglected, allowing head loss from 1 to 2 to be expressed as

$$h_1 = \left(\frac{p_1}{\gamma} + z_1 \right) - \left(\frac{p_2}{\gamma} + z_2 \right). \quad (8-3)$$

It follows that the head loss is independent of the inclination of the column. Darcy related flow rate to head loss and length of column through a proportionality constant referred to as K , the **hydraulic conductivity**, a measure of the ability of the porous media to transmit water. Darcy's law can be stated

$$V = \frac{Q}{A} = -K \frac{dh}{dL}. \quad (8-4)$$

The negative sign indicates that flow of water is in the direction of decreasing head. The Darcy velocity that results from Equation (8-4) is an average discharge velocity through the entire cross section of the column. The actual flow is limited to the pore channels only, so that the seepage velocity V_s is equal to the Darcy velocity divided by porosity:

$$V_s = \frac{Q}{nA}. \quad (8-5)$$

Thus, actual seepage velocities are usually much higher (by a factor of 3) than the Darcy velocities. Seepage velocity should be used for all contaminant transport calculations in ground water.

It should be pointed out that Darcy's law applies to laminar flow in porous media, and experiments indicate that Darcy's law is valid for Reynolds numbers less than 1 and perhaps as high as 10. This represents an upper limit to the validity of Darcy's law, which turns out to be applicable in most ground water systems. Deviations can occur near pumped wells and in fractured aquifer systems with large openings.

Hydraulic Conductivity

The hydraulic conductivity of a soil or rock depends on a variety of physical factors and is an indication of an aquifer's ability to transmit water. Thus, sand aquifers have K values many orders of magnitude larger than clay units. Table 8-3 indicates representative values of hydraulic conductivity for a variety of materials. As can be seen, K can vary over many orders of magnitude in an aquifer that may contain different types of material. Typical values would be 10^{-2} cm/sec for sand, 10^{-4} cm/sec for silt, and 10^{-7} cm/sec for clay aquifers. Thus, velocities and flow rates can also vary over the same range, as expressed by Darcy's law.

Transmissivity is a term often used in ground water hydraulics as applied to confined aquifers. It is defined as the product of K and the average saturated thickness of the aquifer, b . Hydraulic conductivity K is usually expressed in m/day (ft/day), and transmissivity T in m^2/day (ft^2/day). An older unit for T that is still reported in some texts is gal/day/ft. Common conversion factors are contained in Table 8-3.

Table 8-3. Representative Values of Hydraulic Conductivity

Unconsolidated Sediments	Hydraulic Conductivity $K(\text{cm/sec})$
Well-sorted gravel	1 to 10^{-2}
Well-sorted sands and glacial outwash	10^{-1} to 10^{-3}
Silty sands, fine sands	10^{-3} to 10^{-5}
Silt, sandy silt, clayey sands, till	10^{-4} to 10^{-6}
Clay	10^{-6} to 10^{-9}

Note on units: $1 \text{ m/sec} = 1 \times 10^2 \text{ cm/sec} = 3.28 \text{ ft/sec} = 2.12 \times 10^6 \text{ gal/day/ft}^2$.

The **intrinsic permeability** of a rock or soil is a property of the medium only, independent of fluid properties. Intrinsic permeability k can be related to hydraulic conductivity by

$$k = \frac{K\mu}{\rho g}, \quad (8-6)$$

where

$$\begin{aligned} \mu &= \text{dynamic viscosity,} \\ \rho &= \text{fluid density,} \\ g &= \text{gravitational constant.} \end{aligned}$$

Intrinsic permeability k has units of m^2 or darcy, equal to $0.987 (\mu\text{m})^2$; k is often used in the petroleum industry, whereas K is used in ground water hydrology for evaluating aquifer systems.

Determination of Hydraulic Conductivity

Hydraulic conductivity in saturated zones can be determined by a number of techniques in the laboratory as well as in the field. Constant-head and falling-head *permeameters* are used in the laboratory for measuring K and are described in more detail below. In the field, **pump tests**, **slug tests**, and **tracer tests** are available for determination of K . These tests are described in more detail in Sections 8.9 and 8.10 under the general heading of well hydraulics.

A permeameter is used in the laboratory to measure K by maintaining flow through a small column of material and measuring flow rate and head loss. For a constant-head permeameter, Darcy's law can be directly applied to find K , where V is volume flowing in time t through a sample of area A , length L , and with constant head h :

$$K = \frac{VL}{Ath}. \quad (8-7)$$

The falling-head permeameter test consists of measuring the rate of fall of the water level in an attached tube or column and noting that

$$Q = \pi r^2 \frac{dh}{dt}. \quad (8-8)$$

Darcy's law can be written for the sample as

$$Q = \pi r_c^2 K \frac{dh}{dl}. \quad (8-9)$$

After equating and integrating,

$$K = \frac{r^2 L}{r_c^2 t} \ln \left(\frac{h_1}{h_2} \right), \quad (8-10)$$

where L , r , and r_c are the radii of the tube and sample, respectively, and t is the time interval for water to fall from h_1 to h_2 .

In the field, slug tests, pump tests, and tracer tests are preferable for determination of K , since they provide a better estimate of actual field conditions. The slug test for shallow wells operates based on a measurement of decline or recovery of the water level in the well through time. Either the well can be pumped to lower the water level, and allowed to recover in time, or the water level can be increased and allowed to drain out in time (Section 8.10). Hydraulic K is then determined by evaluating the rate of change in the water level with time.

The pump test involves the constant removal of water from a single well and observations of water-level declines at several adjacent wells. In this way, an integrated K value for a portion of the aquifer is obtained. Field methods generally yield different values of K than corresponding laboratory tests performed on cores removed from the aquifer. Thus, field tests are preferable for the accurate determination of aquifer parameters.

Tracer tests involve the injection of inorganic (chloride or bromide tracers) or organic chemicals into a well and the temporal measurement of concentration changes in wells positioned in the direction of ground water flow. Average seepage velocities can be determined by analyzing the breakthrough curves of tracer from the downgradient wells. K can then be determined from Darcy's law.

Anisotropic Aquifers

Most real geologic systems tend to have variations in one or more directions due to the processes of deposition and layering that can occur. In the typical field situation in alluvial deposits, we find the hydraulic conductivity in the vertical direction K_z to be less than the value in the horizontal direction K_x . For the case of a two-layered aquifer of different K in each layer and different thicknesses, we can apply Darcy's law to horizontal flow to show

$$K_x = \frac{K_1 z_1 + K_2 z_2}{z_1 + z_2} \quad (8-11)$$

or, in general,

$$K_x = \frac{\sum K_i z_i}{\sum z_i}, \quad (8-12)$$

where

$$\begin{aligned} K_i &= K \text{ in layer } i, \\ z_i &= \text{thickness of layer } i. \end{aligned}$$

For the case of vertical flow through two layers, q_z is the same flow per unit horizontal area in each layer:

$$dh_1 + dh_2 = \left(\frac{z_1}{K_1} + \frac{z_2}{K_2} \right) q_z, \quad (8-13)$$

but

$$dh_1 + dh_2 = \left(\frac{z_1 + z_2}{K_z} \right) q_z, \quad (8-14)$$

where K_z is the hydraulic conductivity for the entire system. Equating Equations (8-13) and (8-14), we have

$$K_z = \frac{z_1 + z_2}{(z_1/K_1) + (z_2/K_2)} \quad (8-15)$$

or, in general,

$$K_z = \frac{\sum z_i}{\sum z_i/K_i}. \quad (8-16)$$

Ratios of K_x/K_z usually fall in the range of 2 to 10 for alluvium, with values up to 100 where clay layers exist. In actual application to layered systems, it is usually necessary to apply ground water flow models that can properly handle complex geologic strata through numerical simulation. A selected modeling case study for a layered aquifer system is described in Section 8.12.

8.4 FLOW NETS Darcy's law was originally derived in one dimension, but because many ground water problems are really two-dimensional or three-dimensional, methods are available for the determination of flow rate and direction. A specified set of **streamlines** ψ and **equipotential lines** ϕ can be constructed for a given set of boundary conditions to form a **flow net** (Fig. 8-9) in two dimensions. The theory behind ψ and ϕ is presented in more detail in Charbeneau (2000).

For an **isotropic** aquifer, equipotential lines are prepared based on observed water levels in wells penetrating the aquifer. Flow lines are then drawn orthogonally to indicate the direction of flow. For the flow net of Figure 8-9, the hydraulic gradient i is given by

$$i = \frac{dh}{ds}, \quad (8-17)$$

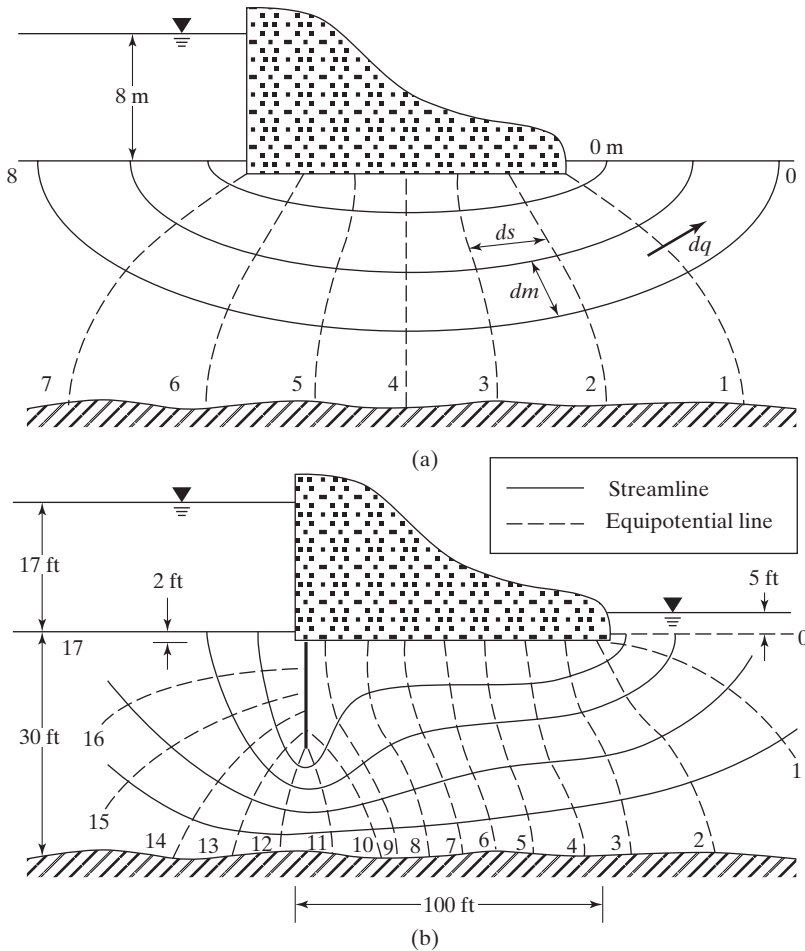
and constant flow q per unit thickness between two adjacent flow lines is

$$q = K \frac{dh}{ds} dm. \quad (8-18)$$

If we assume $ds = dm$ for a square net, then for n squares between two flow lines over which total head is divided ($h = H/n$) and for m divided flow channels,

$$Q' = mq = \frac{KmH}{n}, \quad (8-19)$$

Figure 8-9
Typical flow net
for a dam.



where

Q' = flow per unit width

K = hydraulic conductivity of the aquifer,

m = number of flow channels,

n = number of squares over the direction of flow,

H = total head loss in direction of flow.

Flow nets are useful graphical methods to display streamlines and equipotential lines. Since no flow can cross an impermeable boundary, streamlines must parallel it. Also, streamlines are usually horizontal through high- K material and vertical through low- K material, because of refraction of lines across a boundary between different- K media. It can be shown that

$$\frac{K_1}{K_2} = \frac{\tan \theta_1}{\tan \theta_2} \quad (8-20)$$

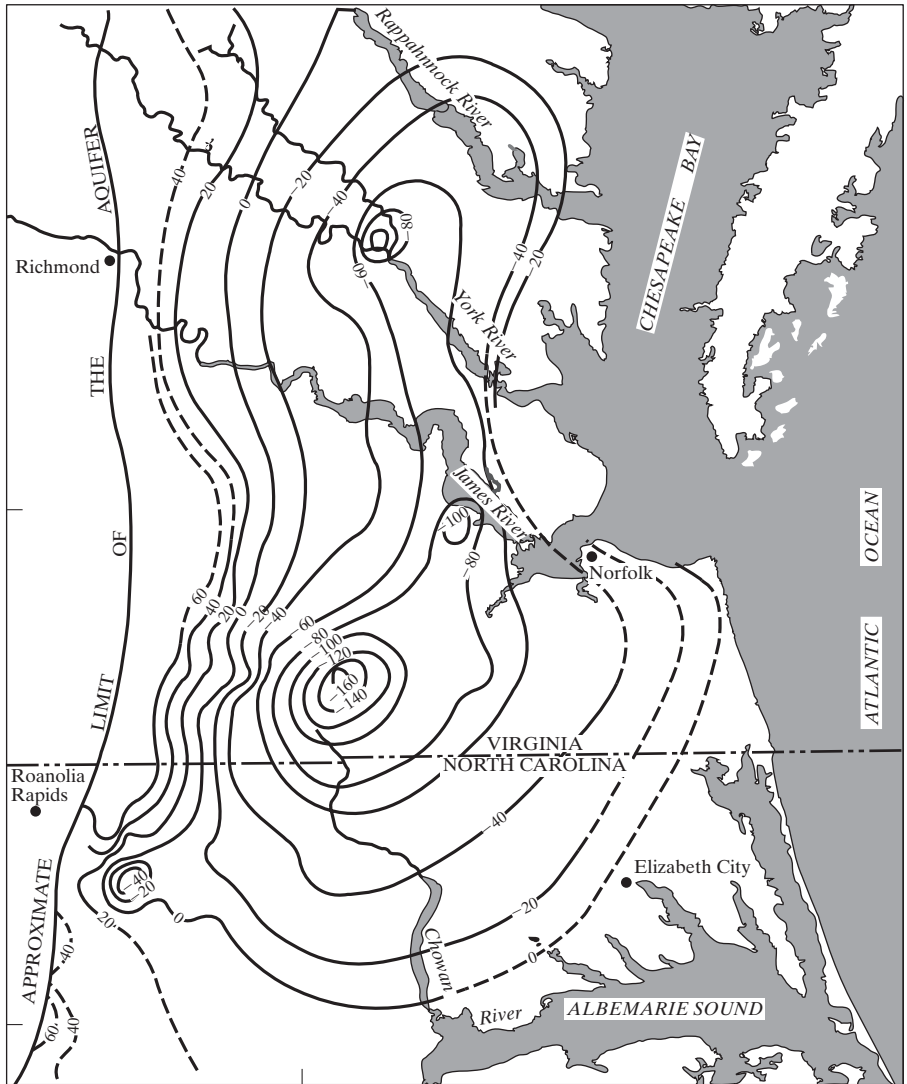


Figure 8-10

Potentiometric surface of lower aquifer in the Atlantic Coastal Plain.

Flow nets can be used to evaluate the effects of pumping on ground water levels and directions of flow. Figure 8-10 depicts the contour map resulting from heavy pumping along the Atlantic coastal plain, over a period of years. Note the cones of depression from several pumping centers. Directions of flow are perpendicular to the equipotential lines, which are lines of constant head.

FLOW NET COMPUTATION**EXAMPLE 8-1**

Compute the total flow seeping under the dam in Figure 8-9(a), where width is 20 m and $K = 10^{-5}$ m/sec. Equation (8-19) is used to provide flow per unit width. From the figure $m = 4$, $n = 8$, $H = 8$ m, and K is given above.

$$Q' = \frac{(10^{-5} \text{ m/sec})(4)(8 \text{ m})}{8},$$

$$Q' = 4 \times 10^{-5} \text{ m}^2/\text{sec} = 3.46 \text{ m}^2/\text{day},$$

$$\text{Total flow: } Q = 3.46 \text{ m}^2/\text{day} (20 \text{ m}) = 69.2 \text{ m}^3/\text{day}.$$

The governing flow equations for ground water are derived in most of the standard texts in the field (Bear, 1979; Freeze and Cherry, 1979; Todd and Mays, 2005; Fetter, 1994; Bedient et al., 1999). The equation of continuity from fluid mechanics is combined with Darcy's law in three dimensions to yield a partial differential equation of flow in porous media, as shown in the next section. Both steady-state and transient flow equations can be derived. Mathematical solutions for specific boundary conditions are well known for the governing ground water flow equation. For complex boundaries and heterogeneous systems, numerical computer solutions must be used (see Bedient et al., 1999).

8.5 GENERAL FLOW EQUATIONS

Steady-State Saturated Flow

Consider a unit volume of porous media (Fig. 8-11) called an elemental control volume. The law of conservation of mass requires that

$$\text{mass in} - \text{mass out} = \text{change in storage per time}.$$

For steady-state conditions, the right-hand side is zero, and the equation of continuity becomes (Fig. 8-11)

$$-\frac{\partial}{\partial x}(\rho V_x) - \frac{\partial}{\partial y}(\rho V_y) - \frac{\partial}{\partial z}(\rho V_z) = 0. \quad (8-21)$$

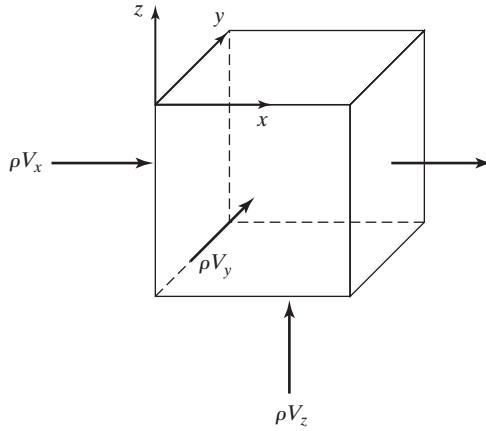
The units of ρV are mass/area/time, as required. For an incompressible fluid, $\rho(x, y, z) = \text{constant}$, and ρ can be divided out of Equation (8-21). Substitution of Darcy's law for V_x , V_y , and V_z yields

$$\frac{\partial}{\partial x} \left(K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left(K_z \frac{\partial h}{\partial z} \right) = 0. \quad (8-22)$$

For an isotropic, homogeneous medium, $K_x = K_y = K_z = K$ and can be divided out of the equation to yield

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0. \quad (8-23)$$

Figure 8-11
Elemental control
volume.



Equation (8-23) is called Laplace's equation and is one of the best-understood partial differential equations. The solution is $h = h(x, y, z)$, the hydraulic head at any point in the flow domain. In two dimensions, the solution is equivalent to the graphical flow nets described in Section 8.4. If there were no variation of h with z , then the equation would reduce to two terms on the left-hand side of Equation (8-23).

Transient Saturated Flow

The transient equation of continuity for a confined aquifer becomes

$$-\frac{\partial}{\partial x}(\rho V_x) - \frac{\partial}{\partial y}(\rho V_y) - \frac{\partial}{\partial z}(\rho V_z) = \frac{\partial}{\partial t}(\rho n) = n \frac{\partial \rho}{\partial t} + \rho \frac{\partial n}{\partial t}. \quad (8-24)$$

The first term on the right-hand side of Equation (8-24) is the mass rate of water produced by an expansion of water under a change in ρ . The second term is the mass rate of water produced by compaction of the porous media (change in n). The first term relates to the compressibility of the fluid β and the second term to the aquifer compressibility α .

Compressibility and effective stress are discussed in detail in Freeze and Cherry (1979), Domenico and Schwartz (1998), and Charbeneau (2000), and will only be briefly reviewed here. The total stress acting on a plane in a saturated porous media is due to the sum of the weight of overlying rock and fluid pressure. The portion of the total stress not borne by the fluid is the effective stress σ_e . Since total stress can be considered constant in most problems, the change in effective stress is equal to the negative of the pressure change in the media, which is related to head change by $dp = \rho g dh$. Thus, a decrease in hydraulic head or pressure results in an increase in effective stress, since $d\sigma_e = -dp$.

The compressibility of water β implies that a change in volume occurs for a given change in stress or pressure, defined as $(-dV/V)/dp$, where dV

is volume change of a given mass of water under a pressure change of dp . The compressibility is approximately constant at $4.4 \times 10^{-10} \text{ m}^2/\text{N}$ for water at usual ground water temperatures.

The compressibility of the porous media or aquifer, α , is related to vertical consolidation for a given change in effective stress, or $\alpha = (db/b)/d\sigma_e$, where b is the vertical dimension. From laboratory studies, α is a function of the applied stress and is dependent on previous loading history. Clays respond differently than sands in this regard, and compaction of clays is largely irreversible for a reduced pressure in the aquifer compared to the response in sands. Land surface subsidence is a good example of aquifer compressibility on a regional scale where clays have been depressured over time.

Freeze and Cherry (1979) indicate that a change in h will produce a change in ρ and n , and the volume of water produced for a unit head decline is S_s , the specific storage. Theoretically, we can show that

$$S_s = \rho g(\alpha + n\beta), \quad (8-25)$$

and the mass rate of water produced is $S_s(\partial h/\partial t)$. Equation (8-24) becomes, after substituting Equation (8-25) and Darcy's law,

$$\frac{\partial}{\partial x} \left(K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left(K_z \frac{\partial h}{\partial z} \right) = S_s \frac{\partial h}{\partial t}. \quad (8-26)$$

For homogeneous and isotropic media,

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S_s}{K} \frac{\partial h}{\partial t}. \quad (8-27)$$

For the special case of a horizontal confined aquifer of thickness b ,

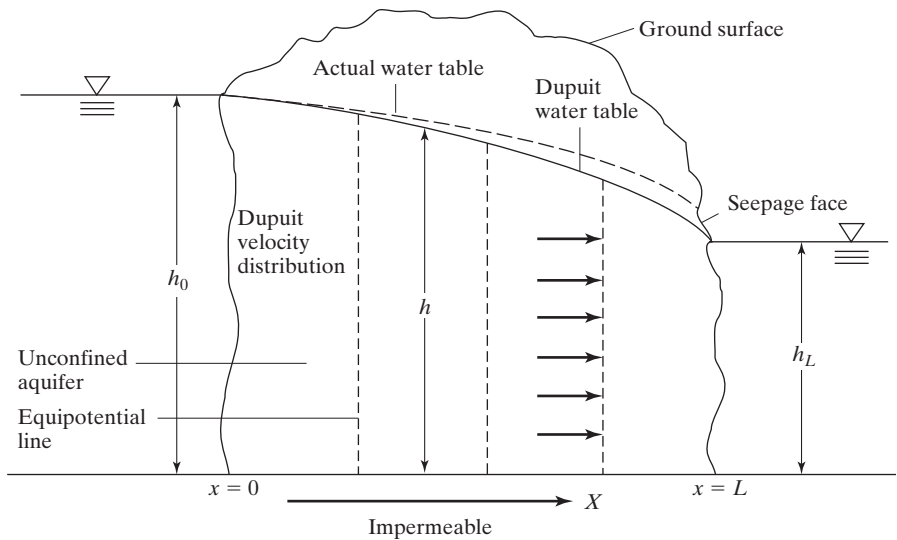
$$\begin{aligned} S &= S_s b, & \text{where } S \text{ is the storativity or storage coefficient} \\ T &= Kb, \\ \nabla^2 h &= \frac{S}{T} \frac{\partial h}{\partial t} & \text{in two dimensions} \end{aligned} \quad (8-28)$$

Solution of Equation (8-28) requires knowledge of S and T to produce $h(x, y)$ over the flow domain. The classical development of Equation (8-28) was first advanced by Jacob (1940) along with considerations of storage concepts.

For the case of unconfined ground water flow, Dupuit developed a theory that allows for a simple solution based on several important assumptions:

1. The water table or free surface is only slightly inclined.
2. Streamlines may be considered horizontal and equipotential lines vertical.
3. Slopes of the free surface and hydraulic gradient are equal.

8.6 DUPUIT EQUATION

**Figure 8-12**

Steady flow in an unconfined aquifer between two water bodies.

Figure 8-12 shows the graphical example of Dupuit's assumptions for essentially one-dimensional flow. The free surface from $x = 0$ to $x = L$ can be derived by considering Darcy's law and the governing one-dimensional equation. Example 8-2 shows the derivation of the Dupuit equations.

EXAMPLE 8-2**DERIVATION OF THE DUPUIT EQUATION**

Derive the equation for one-dimensional flow in an unconfined aquifer, using the Dupuit assumptions (Fig. 8-12).

SOLUTION

Darcy's law gives the one-dimensional flow per unit width as

$$q = -Kh \frac{dh}{dx},$$

where h and x are as defined in Figure 8-12. At steady state, the rate of change of q with distance is zero, or

$$\begin{aligned} \frac{d}{dx} \left[-Kh \frac{dh}{dx} \right] &= 0, \\ -\frac{K}{2} \frac{d^2 h^2}{dx^2} &= 0, \end{aligned}$$

or the governing flow equation becomes

$$\frac{d^2 h^2}{dx^2} = 0.$$

Integration yields

$$h^2 = ax + b,$$

where a and b are constants. Setting the boundary condition $h = h_0$ at $x = 0$,

$$b = h_0^2.$$

Differentiation of $h^2 = ax + b$ gives

$$a = 2h \frac{dh}{dx}.$$

From Darcy's law,

$$h \frac{dh}{dx} = -\frac{q}{K},$$

so, by substitution into the governing flow equation,

$$h^2 = h_0^2 - \frac{2qx}{K}.$$

Setting $h = h_L$ at $x = L$ and neglecting flow across the seepage face yields

$$h_L^2 = h_0^2 - \frac{2qL}{K}.$$

Rearrangement gives

$$q = \frac{K}{2L} (h_0^2 - h_L^2) \quad \text{Dupuit equation}$$

Then, the general equation for the shape of the parabola is

$$h^2 = h_0^2 - \frac{x}{L} (h_0^2 - h_L^2) \quad \text{Dupuit parabola}$$

The derivation of the Dupuit equations in Example 8–2 does not consider recharge to the aquifer. For the case of a system with recharge, the **Dupuit parabola** will take the mounded shape shown in Figure 8–13. The point where $h = h_{\max}$ is known as the **water divide**. At the water divide, $q = 0$, since the gradient is zero. Example 8–3 applies the Dupuit equation for recharge and illustrates the use of the water-divide concept.

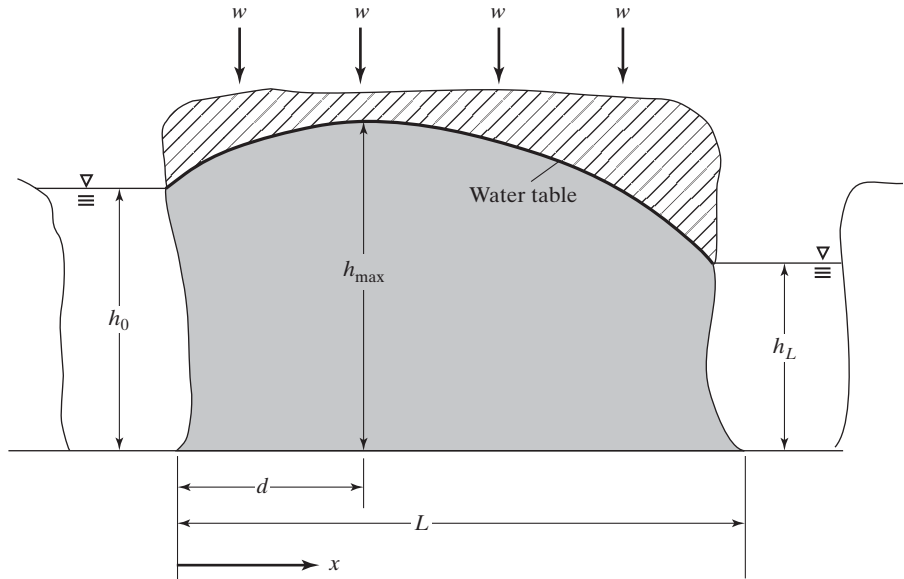


Figure 8-13
Dupuit parabola with recharge.

EXAMPLE 8-3

DUPUIT EQUATION WITH RECHARGE W

- (a) Two rivers located 1000 m apart fully penetrate an aquifer (see Fig. 8-13). The aquifer has a K value of 0.5 m/day. The region receives an average rainfall of 15 cm/yr and evaporation is about 10 cm/yr. Assume that the water elevation in River 1 is 20 m and the water elevation in River 2 is 18 m. Use the Dupuit equation with recharge, and determine the location and height of the water divide.
- (b) What is the daily discharge per m width into River 1 and River 2?

SOLUTION

The Dupuit equations with recharge become

$$q = \frac{K}{2L} (h_0^2 - h_L^2) + W \left(x - \frac{L}{2} \right) \quad \text{for the flow rate per unit width.}$$

$$h^2 = h_0^2 - \frac{x}{L} (h_0^2 - h_L^2) + \frac{Wx}{K} (L - x) \quad \text{for the parabola.}$$

(a) Given

$$\begin{aligned} L &= 1000 \text{ m,} \\ K &= 0.5 \text{ m/day,} \end{aligned}$$

$$h_0 = 20 \text{ m},$$

$$h_L = 18 \text{ m},$$

$$W = 5 \text{ cm/yr} = 1.369 \times 10^{-4} \text{ m/day}.$$

At $x = d$, $q = 0$ (see Fig. 8–13), and d can be found by

$$\begin{aligned} 0 &= \frac{K}{2L} (h_0^2 - h_L^2) + W \left(d - \frac{L}{2} \right), \\ d &= \frac{L}{2} - \frac{L}{2WL} (h_0^2 - h_L^2) \\ &= \frac{1000 \text{ m}}{2} - \frac{(0.5 \text{ m/day}) (20^2 \text{ m}^2 - 18^2 \text{ m}^2)}{(2) (1.369 \times 10^{-4} \text{ m/day}) (1000 \text{ m})} \\ &= 500 \text{ m} - 138.8 \text{ m}, \\ d &= 361.2 \text{ m}. \end{aligned}$$

At $x = 361.2 \text{ m}$ from the edge of River 1, $h = h_{\max}$.

Substituting into the parabola equation, $h_{\max} = 20.9 \text{ m}$.

(b) For discharge into River 1, set $x = 0 \text{ m}$:

$$\begin{aligned} q &= \frac{K}{2L} + (h_0^2 - h_L^2) + W \left(0 - \frac{L}{2} \right) \\ &= \frac{(0.5 \text{ m/day})}{(2) (1000 \text{ m})} (20^2 \text{ m}^2 - 18^2 \text{ m}^2) \\ &\quad + (1.369 \times 10^{-4} \text{ m/day}) (-1000 \text{ m}/2), \\ q &= -0.05 \text{ m}^2/\text{day into River 1}. \end{aligned}$$

The negative sign indicates that flow is in the opposite direction from the x -direction. Therefore, flow is toward River 1 to the left.

For discharge into River 2, set $x = L = 1000 \text{ m}$:

$$\begin{aligned} q &= \frac{K}{2L} + (h_0^2 - h_L^2) + W \left(1000 \text{ m} - \frac{L}{2} \right) \\ &= \frac{(0.5 \text{ m/day})}{(2) (1000 \text{ m})} (20^2 \text{ m}^2 - 18^2 \text{ m}^2) \\ &\quad + (1.369 \times 10^{-4} \text{ m/day}) (1000 \text{ m} - 1000 \text{ m}/2), \\ q &= 0.087 \text{ m}^2/\text{day into River 2}. \end{aligned}$$

8.7
STREAMLINES
AND
EQUIPOTENTIAL
LINES

The equation of continuity for steady, incompressible, isotropic flow in two dimensions is

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0.$$

The governing steady-state ground water flow equation is

$$\nabla^2 h = \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0. \quad (8-29)$$

Potential theory applies to flow fields where the flux may be derived from the gradient of a scalar field called a potential field. The common path of a fluid particle is called a **flow line** or a **streamline**. Charbeneau (2000) presents an excellent discussion of potential theory and stream functions for a variety of common ground water conditions.

The **potential function** ϕ is a scalar function and can be written

$$\phi(x, y) = -K(z + \rho/\gamma) + c, \quad (8-30)$$

where K and c are assumed constant. From Darcy's law in two dimensions,

$$u = \frac{\partial \phi}{\partial x}, \quad v = \frac{\partial \phi}{\partial y}. \quad (8-31)$$

Using Equation (8-29) in two dimensions, we have

$$\nabla^2 \phi = \frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0. \quad (8-32)$$

where $\phi(x, y) = \text{constant}$ represents a family of equipotential curves on a two-dimensional surface. It can be shown that the **stream function** $\psi(x, y)$ is a constant along a flow line and is orthogonal to $\phi(x, y) = \text{constant}$. Both functions satisfy the equation of continuity and Laplace's equation. The stream function $\psi(x, y)$ is defined by

$$u = \frac{\partial \psi}{\partial y}, \quad v = -\frac{\partial \psi}{\partial x}. \quad (8-33)$$

Combining Equations (8-31) and (8-33), the Cauchy–Riemann equations become

$$\frac{\partial \psi}{\partial x} = \frac{\partial \psi}{\partial y}, \quad \frac{\partial \psi}{\partial y} = -\frac{\partial \psi}{\partial x}. \quad (8-34)$$

If ψ is constant along a flow line, then for any displacement along the flow line,

$$d\psi = \frac{\partial \psi}{\partial x} dx + \frac{\partial \psi}{\partial y} dy = 0,$$

and it can be shown that ψ must also satisfy Laplace's equation

$$\nabla^2 \psi = \frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} = 0. \quad (8-35)$$

It can be easily shown that $\psi(x, y)$ and $\phi(x, y)$ are orthogonal (at right angles) for homogeneous, isotropic flow and essentially result in the flow nets described earlier (see Fig. 8-7). Also, the flow between two streamlines is given by the difference in their stream function values, and the spacing between streamlines reveals the relative magnitude of the flow between them (Bear, 1979; Charbeneau, 2000).

The nonlinear nature of soil moisture relationships greatly complicates analyses in the unsaturated zone (see Section 2.9 on infiltration theory). Hydraulic conductivity $K(\theta)$ in the unsaturated zone above the water table relates velocity and hydraulic gradient in Darcy's law. **Moisture content** θ is defined as the ratio of the volume of water to the total volume of a unit of porous media. To complicate the analysis of unsaturated flow, the moisture content θ and the hydraulic conductivity K are functions of the capillary suction ψ . Also, it has been observed experimentally that the $\theta - \psi$ relationships differ significantly for different types of soil. Figure 8-4 shows the characteristic drying and wetting curves that occur in soils that are draining water or receiving infiltration of water (see Fig. 2-19).

The water table defines the boundary between the unsaturated and saturated zones and is defined by the surface on which the fluid pressure P is exactly atmospheric, or $P = 0$. Hence, the total hydraulic head $\varphi = \psi + z$, where $\psi = P/\rho g$, the pressure head. For saturated ground water flow, θ equals the porosity of the sample n , defined as the ratio of volume of voids to total volume of sample; for unsaturated flow above a water table, $\theta < n$.

Darcy's law is used with the unsaturated value for K and can be written [Eq. (8-4)]

$$v = -K(\theta) \frac{\partial h}{\partial z},$$

where

- v = Darcy velocity,
- z = depth below surface,
- h = potential or head = $z + \psi$,
- ψ = tension or suction,
- $K(\theta)$ = unsaturated hydraulic conductivity,
- θ = volumetric moisture content.

Near the water table a capillary fringe can occur where ψ is a small negative pressure corresponding to the air entry pressure. This capillary zone is small for sandy soils but can be up to two meters deep for fine-grained soils. By definition, pressure head is negative (under tension) at all points above the water table and is positive for points below the water table. The value of ψ is greater than zero in the saturated zone below the water table and equals zero at the water table. Soil physicists refer to $\psi < 0$ as the tension head or

8.8 UNSATURATED FLOW

capillary suction head, and it can be measured in the laboratory or field by an instrument called a tensiometer.

To summarize the properties of the unsaturated zone as compared to the saturated zone, Freeze and Cherry (1979) state for the unsaturated zone (vadose zone)

1. It occurs above the water table and above the capillary fringe.
2. The soil pores are only partially filled with water; the moisture content θ is less than the porosity n .
3. The fluid pressure p is less than atmospheric; the pressure head ψ is less than zero.
4. The hydraulic head h must be measured with a tensiometer.
5. The hydraulic conductivity K and the moisture content θ are both functions of the pressure head ψ .

More details on the unsaturated zone can be found in other textbooks, where both flow and transport in the unsaturated zone are described along with applications of analytical and numerical methods. However, the unsaturated zone still remains a very complex environment in which detailed results are difficult to measure and predict. The interested student should consult such books as Guymon (1994), Bedient et al. (1999), and Charbeneau (2000) for more details.

8.9
STEADY-STATE
WELL
HYDRAULICS

The case of steady flow to a well implies that the variation of head occurs only in space and not in time. The governing equations presented in Section 8.5 can be solved for pumping wells in unconfined or confined aquifers under steady or unsteady conditions. Boundary conditions must be kept relatively simple, and aquifers must be assumed to be homogeneous and isotropic in each layer. More complex geometries can be handled by numerical simulation models in two or three dimensions (Section 8.12).

Steady One-Dimensional Flow

For the case of ground water flow in the x -direction in a confined aquifer, the governing equation becomes

$$\frac{d^2h}{dx^2} = 0 \quad (8-36)$$

and has the solution

$$h = \frac{-vx}{K}, \quad (8-37)$$

where $h = 0$ and $x = 0$ and $dh/dx = -v/K$ according to Darcy's law. This states that head varies linearly with flow in the x -direction. If plotted, the solution would appear as the simple flow net in Figure 8-7 with parallel streamlines in only one direction, from higher head toward lower head.

The simplest case of steady one-dimensional flow in an unconfined aquifer was presented in Section 8.6, using Dupuit's assumptions. The resulting variation of head with x is called the Dupuit parabola and represents the approximate shape of the water table for relatively flat slopes. In the presence of steep slopes near wells, the Dupuit approximation may be in error, and more sophisticated computer methods should be used.

Steady Radial Flow to a Well—Confined

The **drawdown** curve, or **cone of depression**, varies with distance from a pumping well in a confined aquifer (Fig. 8-14). The flow is assumed two-dimensional for a completely penetrating well in a homogeneous, isotropic aquifer of unlimited extent. For horizontal flow, the above assumptions apply and Q at any r equals, from Darcy's law,

$$Q = -2\pi r b K \frac{dh}{dr} \quad (8-38)$$

for steady radial flow to a well. Integrating after separation of variables, with $h = h_w$ at $r = r_w$ at the well, yields

$$Q = 2\pi K b \frac{h - h_w}{\ln(r/r_w)}. \quad (8-39)$$

Equation (8-39) shows that h increases indefinitely with increasing r , yet the maximum head is h_0 for Figure 8-14. Near the well the relationship holds and can be rearranged to yield an estimate for transmissivity T ,

$$T = Kb = \frac{Q}{2\pi(h_2 - h_1)} \ln \frac{r_2}{r_1}, \quad (8-40)$$

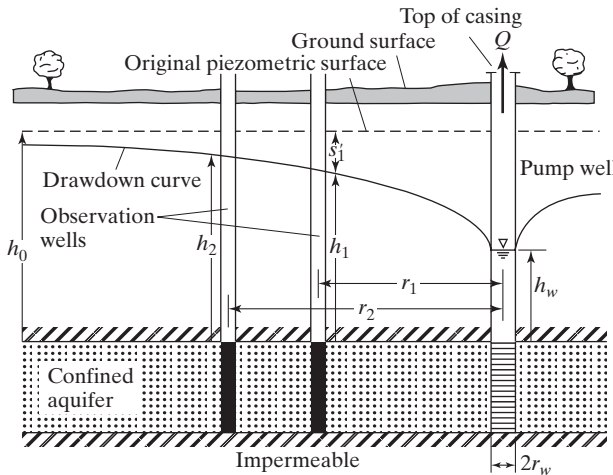


Figure 8-14
Radial flow to a well penetrating an extensive confined aquifer.

by observing heads h_1 and h_2 at two adjacent observation wells located at r_1 and r_2 , respectively, from the pumping well. In practice, it is often necessary to use unsteady-state analyses because of the long times required to reach steady state.

EXAMPLE 8–4**DETERMINATION OF K AND T IN A CONFINED AQUIFER**

A well is constructed to pump water from a confined aquifer. Two observation wells, MW1 and MW2, are constructed at distances of 100 m and 1000 m, respectively. Water is pumped from the pumping well at a rate of $0.2 \text{ m}^3/\text{min}$. At steady state, drawdown s' is observed as 2 m in MW2 and 8 m in MW1. Note that drawdown is greater (8 m) for well MW1 located closest to the pumping well. Determine the hydraulic conductivity K and transmissivity T if the aquifer is 20 m thick.

SOLUTION

Given

$$Q = 0.2 \text{ m}^3/\text{min},$$

$$r_2 = 1000 \text{ m},$$

$$r_1 = 100 \text{ m},$$

$$s'_2 = 2 \text{ m},$$

$$s'_1 = 8 \text{ m},$$

$$b = 20 \text{ m}.$$

Equation (8–40) gives

$$T = Kb = \frac{Q}{2\pi(h_2 - h_1)} \ln \left(\frac{r_2}{r_1} \right).$$

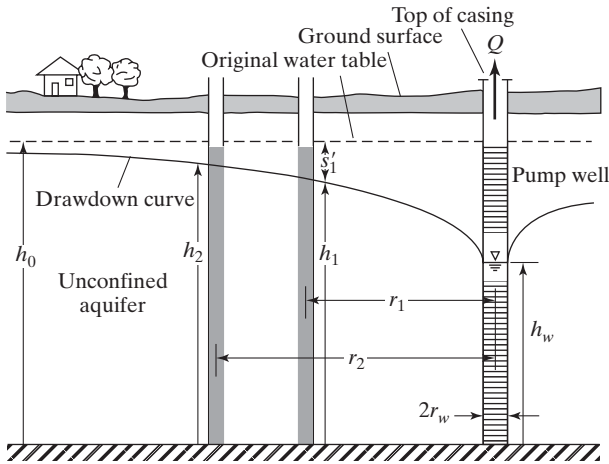
Knowing that $s'_1 = h_0 - h_1$ and $s'_2 = h_0 - h_2$, we have

$$\begin{aligned} T = Kb &= \frac{Q}{2\pi(s'_1 - s'_2)} \left[\ln \left(\frac{r_2}{r_1} \right) \right] \\ &= \frac{0.2 \text{ m}^3/\text{min}}{(2\pi)(8 \text{ m} - 2 \text{ m})} \ln \left(\frac{1000 \text{ m}}{100 \text{ m}} \right), \end{aligned}$$

$$T = 0.0122 \text{ m}^2/\text{min} = 2.04 \text{ cm}^2/\text{sec}.$$

Then,

$$\begin{aligned} K &= T/b \\ &= (2.04 \text{ cm}^2/\text{s})/(20 \text{ m})(100 \text{ cm}/1 \text{ m}), \\ K &= 1.02 \times 10^{-3} \text{ cm/sec}. \end{aligned}$$

**Figure 8-15**

Radial flow to a well penetrating an unconfined aquifer.

Steady Radial Flow to a Well—Unconfined

Applying Darcy's law for radial flow in an unconfined, homogeneous, isotropic, and horizontal aquifer and using Dupuit's assumptions (Fig. 8-15),

$$Q = -2\pi rKh \frac{dh}{dr}. \quad (8-41)$$

Integrating, as before,

$$Q = \pi K \frac{h_2^2 - h_1^2}{\ln(r_2/r_1)}. \quad (8-42)$$

Solving for K ,

$$K = \frac{Q}{\pi(h_2^2 - h_1^2)} \ln \frac{r_2}{r_1}, \quad (8-43)$$

where heads h_1 and h_2 are observed at adjacent wells located distances r_1 and r_2 from the pumping well, respectively.

DETERMINATION OF K IN AN UNCONFINED AQUIFER

EXAMPLE 8-5

A fully penetrating well discharges 75 gpm from an unconfined aquifer. The original water table was recorded as 35 ft. After a long time period the water table was recorded as 20 ft MSL in an observation well located 75 ft away and 34 ft MSL at an observation well located 2000 ft away. Determine the hydraulic conductivity of this aquifer in ft/s.

SOLUTION**Given**

$$Q = 75 \text{ gpm},$$

$$r_2 = 2000 \text{ ft},$$

$$r_1 = 75 \text{ ft},$$

$$h_2 = 34 \text{ ft},$$

$$h_1 = 20 \text{ ft}.$$

Equation (8-43) gives

$$\begin{aligned} K &= \frac{Q}{\pi(h_2^2 - h_1^2)} \ln\left(\frac{r_2}{r_1}\right) \\ &= \frac{(75 \text{ gpm})(0.134 \text{ ft}^3/\text{gal})(1 \text{ min}/60 \text{ s})}{(\pi)(34^2 \text{ ft}^2 - 20^2 \text{ ft}^2)} \ln\left(\frac{2000 \text{ ft}}{75 \text{ ft}}\right), \\ K &= 2.32 \times 10^{-4} \text{ ft/sec}. \end{aligned}$$

Well Pumping from a Leaky Aquifer System

A typical problem involves the case of a well pumping at steady state from a deeper aquifer, which has a shallow water-table aquifer ($k = 10 \text{ ft/day}$) lying above (Fig. 8-16). An aquitard ($K = 0.3 \text{ ft/day}$) separates the overlying unconfined from the deeper leaky confined aquifer. The level of the water table is above the level of the piezometric surface, and so water moves vertically downward through the aquitard, as shown. We write Darcy's law from point A to B , over a distance of 29 ft, and use C as the location of the datum. With the dimensions and parameter values shown in Figure 8-16, we obtain

$$v = K \frac{dh}{dl} = 10 \frac{34 - h_b}{29}. \quad (8-44)$$

By writing Darcy's law from point B to C , over a distance of 5 ft, we obtain

$$v = K \frac{dh}{dl} = 0.3 \frac{h_b - 30}{5}. \quad (8-45)$$

By setting the two velocities equal at steady state, we can solve for h_b

$$h_b = 33.4 \text{ ft} \quad \text{measured above point } C, \quad (8-46)$$

where it can be seen that the head at point B is slightly less than the head at A , the unconfined aquifer level. Thus, there is a head loss associated with vertical downward movement through the shallow aquifer of 0.6 ft.

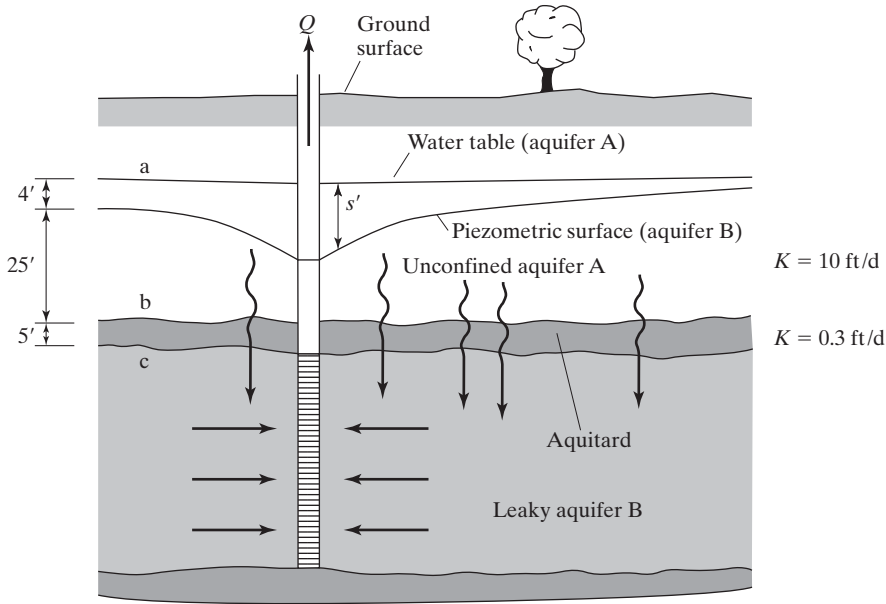


Figure 8-16
Application of Darcy's law for leaky aquifer system.

If the above aquifer were to be pumped from the lower aquifer, then additional leakage would occur from the upper to the lower due to the increase in head difference. If the upper aquifer were to be pumped, then one could reverse the flow of water from lower toward upper. The case study in Section 8.12 illustrates the concept of recharge and its relationship to water levels due to overpumping in a layered limestone aquifer system in Florida.

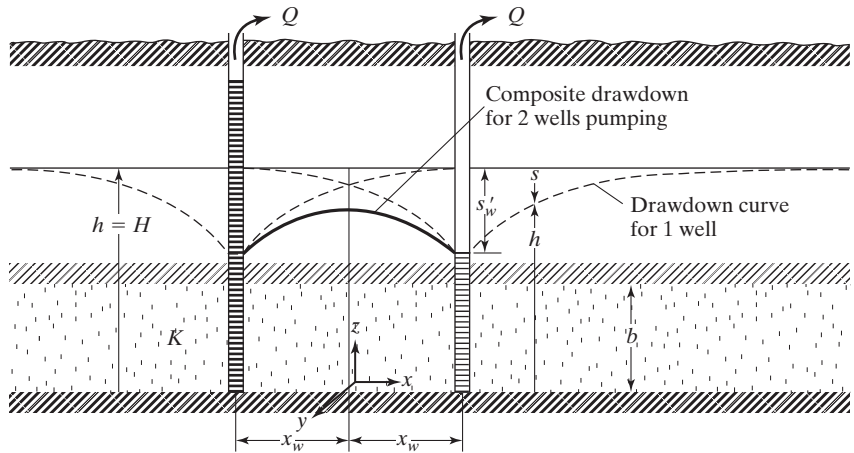
Multiple-Well Systems

For multiple wells with drawdowns that overlap, the principle of linear superposition can be used. This principle implies that the drawdown level at any point in the area of influence of several pumping wells is equal to the sum of drawdowns from each contributing well in confined or unconfined aquifers (Fig. 8-17). Note that the drawdown in between two pumping wells with the same pumping rate is greater than that from a single well.

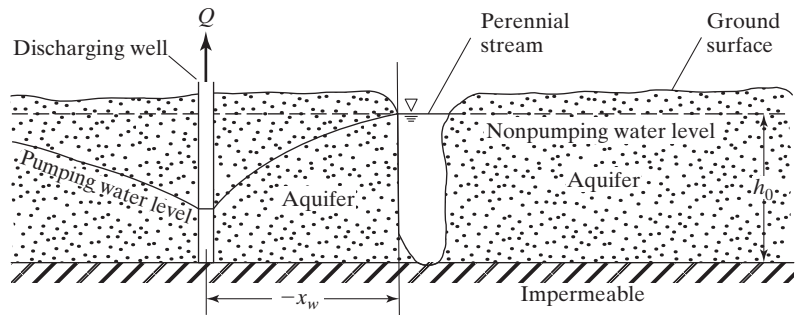
The same principle applies for well flow near a boundary. Figure 8-18 shows the cases for a well pumping near a fixed-head stream and near an impermeable boundary. Image wells placed on the other side of the boundary at a distance x_w can be used to represent the equivalent hydraulic condition. In case A, the image well is recharging at the same rate Q , and in case B it is pumping at rate Q . The summation of drawdowns from the original pumping well and the image well provides a correct boundary condition at

Figure 8-17

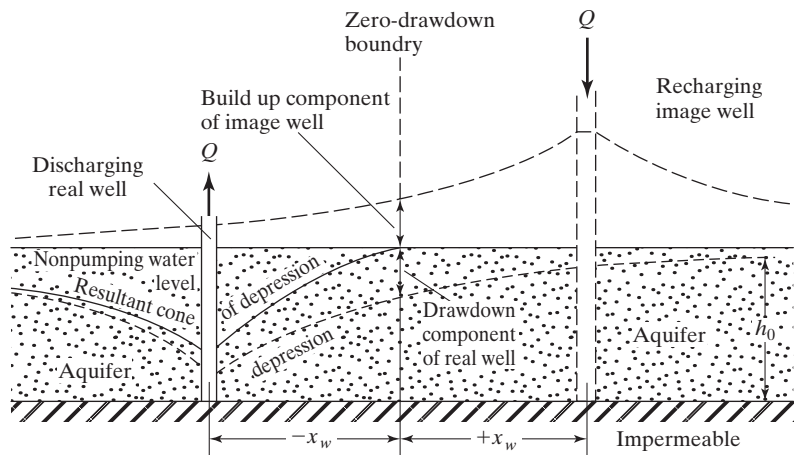
Individual and composite drawdown curves for two wells in a line.

**Figure 8-18a**

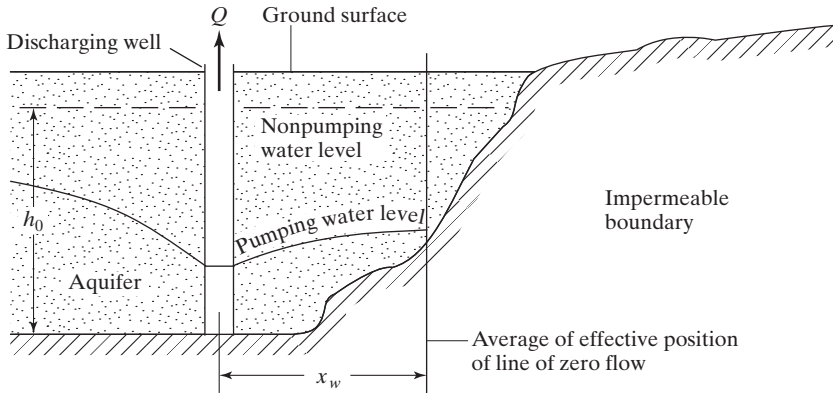
Sectional views.
(1) Discharging well near a perennial stream.
(2) Equivalent hydraulic system in an aquifer of infinite areal extent.



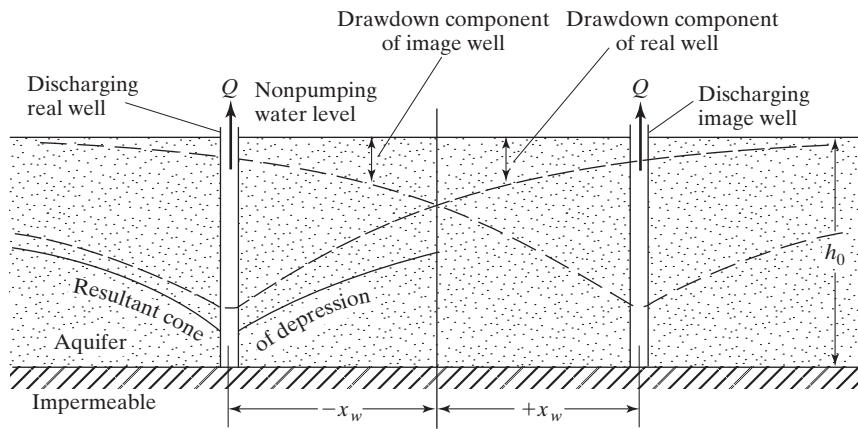
(1)



(2)



(1)



(2)

Figure 8-18b

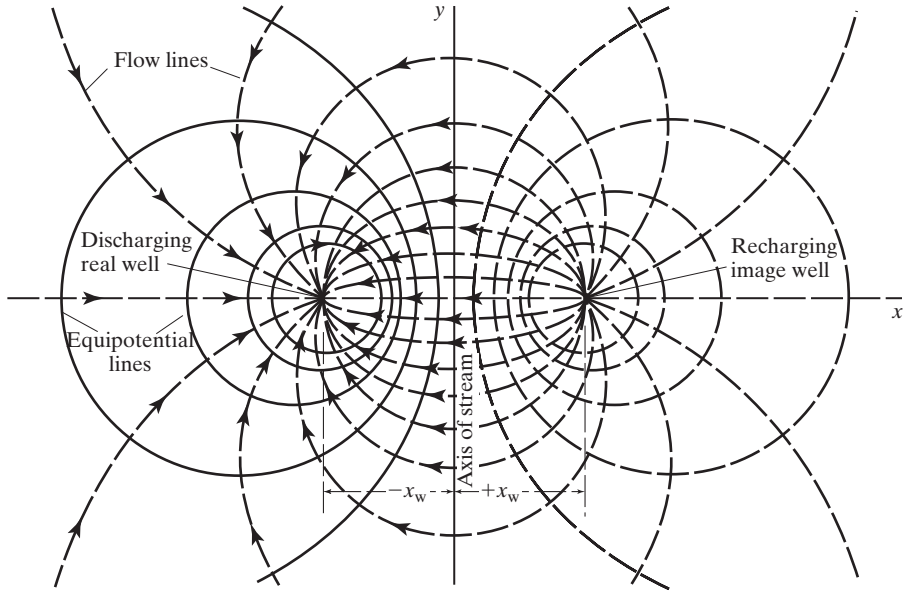
(1) Discharging well near an impermeable boundary. (2) Equivalent hydraulic system in an aquifer of infinite areal extent. (After Ferris et al., 1962.)

any distance r from the well. Thus, the use of image wells allows an aquifer of finite extent to be transformed into an infinite aquifer so that closed-form solution methods can be applied.

Figure 8-19 shows a flow net for a pumping well and a recharging image well and indicates a line of constant head between the two wells. The steady-state drawdown s' at any point (x, y) , where y is the axis of symmetry, is given by

$$s' = \frac{Q}{4\pi T} \ln \frac{(x + x_w)^2 + (y - y_w)^2}{(x - x_w)^2 + (y - y_w)^2}, \quad (8-47)$$

where $\pm x_w$ and $y_w = 0$ are the locations of the recharge and discharge wells for the case shown in Figure 8-19.

**Figure 8-19**

Flow net for a discharging real well and a recharging image well. (After Ferris et al., 1962.)

8.10 UNSTEADY WELL HYDRAULICS

The Theis Method of Solution

As a well penetrating a confined aquifer of infinite extent is pumped at a constant rate, a drawdown occurs radially, extending from the well. The rate of decline of head times the storage coefficient summed over the area of influence equals the discharge. The rate of decline decreases continuously as the area of influence expands.

The governing ground water flow equation [Eq. (8-27)] in plane polar coordinates is

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \frac{\partial h}{\partial r} = \frac{S}{T} \frac{\partial h}{\partial t}, \quad (8-48)$$

where

- h = head,
- r = radial distance,
- S = storage coefficient,
- T = transmissivity.

Theis (1935) obtained a solution for Equation (8-48) by assuming that the well is a mathematical sink of constant strength and by using boundary conditions $h = h_0$ for $t = 0$ and $h \rightarrow h_0$ as $r \rightarrow \infty$ for $t \geq 0$:

$$s' = \frac{Q}{4\pi T} \int_u^\infty \frac{e^{-u} du}{u} = \frac{Q}{4\pi T} W(u), \quad (8-49)$$

where s' is drawdown, Q is discharge at the well, and

$$u = \frac{r^2 S}{4Tt}. \quad (8-50)$$

Equation (8-49) is known as the nonequilibrium, or Theis, equation. The integral is written as $W(u)$ and is known as the exponential integral, or well function, which can be expanded as a series:

$$W(u) = -0.5772 - \ln(u) + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} - \frac{u^4}{4 \cdot 4!} + \cdots \quad (8-51)$$

The equation can be used to obtain aquifer constants S and T by means of pumping tests at fully penetrating wells. It is widely used because a value of S can be determined, only one observation well and a relatively short pumping period are required, and large portions of the flow field can be sampled with one test.

The assumptions inherent in the Theis equation should be included, since they are often overlooked:

1. The aquifer is homogeneous, isotropic, uniformly thick, and of infinite areal extent.
2. Prior to pumping, the piezometric surface is horizontal.
3. The fully penetrating well is pumped at a constant rate.
4. Flow is horizontal within the aquifer.
5. Storage within the well can be neglected.
6. Water removed from storage responds instantaneously with a declining head.

These assumptions are seldom completely satisfied for a field problem, but the method still provides one of the most useful and accurate techniques for aquifer characterization. The complete Theis solution requires the graphical solution of two equations with four unknowns:

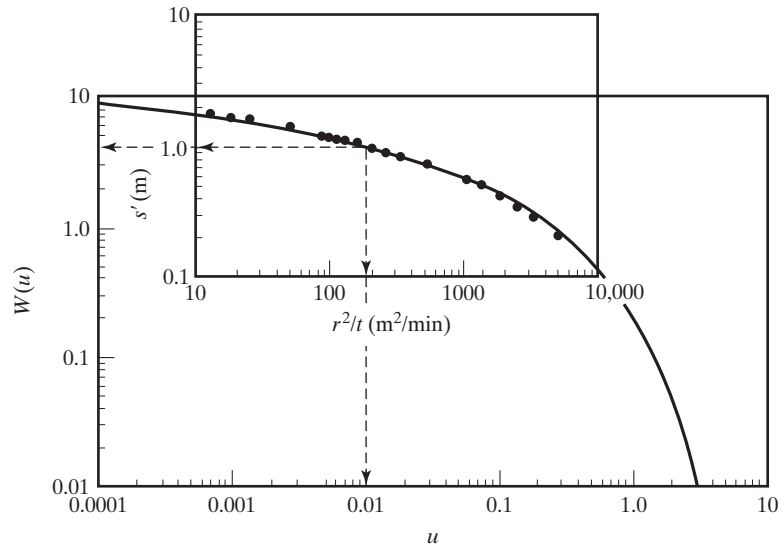
$$s' = \frac{Q}{4\pi T} W(u), \quad (8-52)$$

$$\frac{r^2}{t} = \left(\frac{4T}{S} \right) u. \quad (8-53)$$

The relation between $W(u)$ and u must be the same as that between s' and r^2/t , because all other terms are constants in the equations. Theis suggested a solution based on graphical superposition. Example 8-6 indicates how a plot of $W(u)$ vs. u , called a type curve, is superimposed over observed time-drawdown data while keeping the coordinate axes parallel. The two plots are adjusted until a position is found by trial, such that most of the observed data fall on a segment of the type curve. Any convenient point is selected, and values of $W(u)$, u , s' , and r^2/t are used in Equations (8-52) and (8-53) to determine S and T (see Fig. 8-20).

Figure 8–20

Theis method of superposition for solution of the nonequilibrium equation.



It is also possible to use Theis's solution for the case where several wells are sampled for drawdown simultaneously near a pumped well.

Distance–drawdown data are then fitted to the type curve similar to the method just outlined.

EXAMPLE 8–6**DETERMINATION OF T AND S BY THE THEIS METHOD**

A fully penetrating well in a 25-m-thick confined aquifer is pumped at a rate of $0.2 \text{ m}^3/\text{s}$ for 1000 min. Drawdown is recorded vs. time at an observation well located 100 m away. Compute the transmissivity and storativity using the Theis method.

Time (min)	s' (m)	Time (min)	s' (m)
1	0.11	60	1.02
2	0.20	70	1.05
3	0.28	80	1.08
4	0.34	90	1.11
6	0.44	100	1.15
8	0.50	200	1.35
10	0.54	400	1.55
20	0.71	600	1.61
30	0.82	800	1.75
40	0.85	1000	1.80
50	0.92		

A plot of s' vs. r^2/t is made on log-log paper. This is superimposed on a plot of $W(u)$ vs. u , which is also on log-log paper. A point is chosen at some convenient point on the matched curve, and values for s' , r^2/t , $W(u)$, and u are read (see Fig. 8-20 and the accompanying table of values for the well function).

By overlaying the data with the well function curve, one obtains the best fit plot of the data, and then determines four values from the plot,

$$\begin{aligned} r^2/t &= 180 \text{ m}^2/\text{min}, \\ s' &= 1.0 \text{ m}, \\ u &= 0.01, \\ W(u) &= 4.0. \end{aligned}$$

Solving Equation (8-52) gives an estimate for T based on the pump test data:

$$\begin{aligned} s' &= \frac{Q}{4\pi T} W(u), \\ T &= QW(u)/4\pi s', \\ T &= \frac{(0.2 \text{ m}^3/\text{s})/(4.0)}{(4\pi)(1.0 \text{ m})}, \\ T &= 6.37 \times 10^{-2} \text{ m}^2/\text{sec} \end{aligned}$$

Equation (8-53) gives

$$\begin{aligned} r^2/t &= \frac{4Tu}{S}, \\ S &= \frac{4Tu}{r^2/t} \\ &= (4)(6.37 \times 10^{-2} \text{ m}^2/\text{s})(0.01)/(180 \text{ m}^2/\text{min})(1 \text{ min}/60 \text{ s}), \\ S &= 8.49 \times 10^{-4}. \end{aligned}$$

Values of the Function $W(u)$ for Various Values of u

u	$W(u)$	u	$W(u)$	u	$W(u)$	u	$W(u)$
$1 \cdot 10^{-10}$	22.45	$7 \cdot 10^{-8}$	15.90	$4 \cdot 10^{-5}$	9.55	$1 \cdot 10^{-2}$	4.04
2	21.76	8	15.76	5	9.33	2	3.35
3	21.35	9	15.65	6	9.14	3	2.96
4	21.06	$1 \cdot 10^{-7}$	15.54	7	8.99	4	2.68
5	20.84	2	14.85	8	8.86	5	2.47
6	20.66	3	14.44	9	8.74	6	2.30
7	20.50	4	14.15	$1 \cdot 10^{-4}$	8.63	7	2.15
8	20.37	5	13.93	2	7.94	8	2.03

(Continued)

SOLUTION

u	$W(u)$	u	$W(u)$	u	$W(u)$	u	$W(u)$
9	20.25	6	13.75	3	7.53	9	1.92
$1 \cdot 10^{-9}$	20.15	7	13.60	4	7.25	$1 \cdot 10^{-1}$	1.823
2	19.45	8	13.46	5	7.02	2	1.223
3	19.05	9	13.34	6	6.84	3	0.906
4	18.76	$1 \cdot 10^{-6}$	13.24	7	6.69	4	0.702
5	18.54	2	12.55	8	6.55	5	0.560
6	18.35	3	12.14	9	6.44	6	0.454
7	18.20	4	11.85	$1 \cdot 10^{-3}$	6.33	7	0.374
8	18.07	5	11.63	2	5.64	8	0.311
9	17.95	6	11.45	3	5.23	9	0.260
$1 \cdot 10^{-8}$	17.84	7	11.29	4	4.95	$1 \cdot 10^0$	0.219
2	17.15	8	11.16	5	4.73	2	0.049
3	16.74	9	11.04	6	4.54	3	0.013
4	16.46	$1 \cdot 10^{-5}$	10.94	7	4.39	4	0.004
5	16.23	2	10.24	8	4.26	5	0.001
6	16.05	3	9.84	9	4.14		

Cooper–Jacob Method of Solution

Cooper and Jacob (1946) noted that for small values of r and large values of t , the parameter u in Equation (8–49) becomes very small, so that the well function can be approximated by

$$s' = \frac{Q}{4\pi T} \left[-0.5772 - \ln \left(\frac{r^2 S}{4Tt} \right) \right]. \quad (8-54)$$

Further arrangement and conversion to decimal logs yields

$$s' = \frac{2.30Q}{4\pi T} \log \left(\frac{2.25Tt}{r^2 S} \right). \quad (8-55)$$

Thus, a plot of drawdown s' vs. $\log t$ forms a straight line, as shown in Figure 8–21, and a projection of the line to $s' = 0$, where $t = t_0$, yields

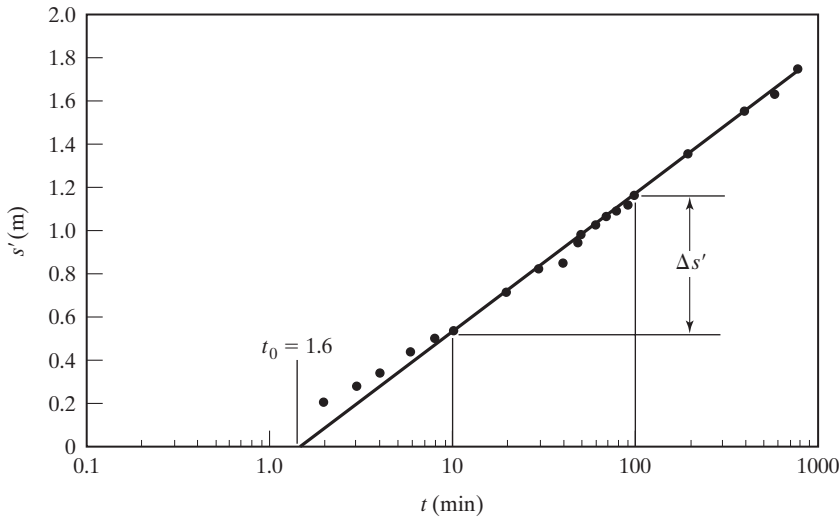
$$0 = \frac{2.30Q}{4\pi T} \log \left(\frac{2.25Tt_0}{r^2 S} \right), \quad (8-56)$$

and it follows that since $\log(1) = 0$, one can solve for S from the argument of the log to get

$$S = \left(\frac{2.25Tt_0}{r^2} \right). \quad (8-57)$$

Finally, by replacing s' by $\Delta s'$, where $\Delta s'$ is the drawdown difference of data per log cycle of t , Equation (8–55) becomes

$$T = \left(\frac{2.3Q}{4\pi \Delta s'} \right). \quad (8-58)$$

**Figure 8-21**

Cooper–Jacob method of analysis.

The Cooper–Jacob method solves first for T with Equation (8-58) and then for S with Equation (8-57) and is applicable for small values of u (less than 0.01). Calculations with the Theis method were presented earlier, and Example 8-7 demonstrates the use of the Cooper–Jacob method.

DETERMINATION OF T AND S BY THE COOPER–JACOB METHOD

EXAMPLE 8-7

Using the data given in Example 8-6, determine the transmissivity and storativity of the 25-m-thick confined aquifer, using the Cooper–Jacob method.

Values of s' and t are plotted on semilog paper with the t -axis logarithmic (see Fig. 8-21). A line is fitted through the later time periods and is projected back to a point where $s' = 0$. This point determines t_0 . $\Delta s'$ is measured over one log cycle of t .

SOLUTION

From the plot,

$$t_0 = 1.6 \text{ min,}$$

$$\Delta s' = 0.65 \text{ m.}$$

Equation (8-58) can be used to provide an estimate for T , the transmissivity,

$$T = \frac{2.3Q}{4\pi \Delta s'}$$

$$= \frac{(2.3)(0.2 \text{ m}^3/\text{s})}{(4\pi)(0.65 \text{ m})},$$

$$T = 5.63 \times 10^{-2} \text{ m}^2/\text{sec.}$$

Then Equation (8-57) provides the value for S , the storage coefficient,

$$\begin{aligned} S &= \frac{2.25 T t_0}{r^2} \\ &= \frac{(2.25) (5.63 \times 10^{-2} \text{ m}^2/\text{s}) (1.6 \text{ min}) (60 \text{ s}/1 \text{ min})}{100^2 \text{ m}^2}, \\ S &= 1.22 \times 10^{-3}. \end{aligned}$$

Slug Tests

Slug tests involve the use of a single well for the determination of aquifer formation constants. Rather than pumping the well for a period of time, as described in the previous section, a volume of water is suddenly removed or added to the well casing and observations of recovery or drawdown are noted through time. By careful evaluation of the drawdown curve and knowledge of the well screen geometry, it is possible to derive K or T for an aquifer.

Typical procedure for a slug test requires use of a rod of slightly smaller diameter than the well casing or a pump to evacuate the well casing. The simplest slug-test method in a piezometer was published by Hvorslev (1951), who used the recovery of water level over time to calculate hydraulic conductivity of the porous media. Hvorslev's method relates the flow $q(t)$ at the piezometer at any time to the hydraulic conductivity and the unrecovered head distance, $H_0 - h$, in Figure 8-22, by

$$q(t) = \pi r^2 \frac{dh}{dt} = FK(H_0 - h), \quad (8-59)$$

where F is a factor that depends on the shape and dimensions of the piezometer intake. If $q = q_0$ at $t = 0$, then $q(t)$ will decrease toward zero as time increases. Hvorslev defined the basic time lag

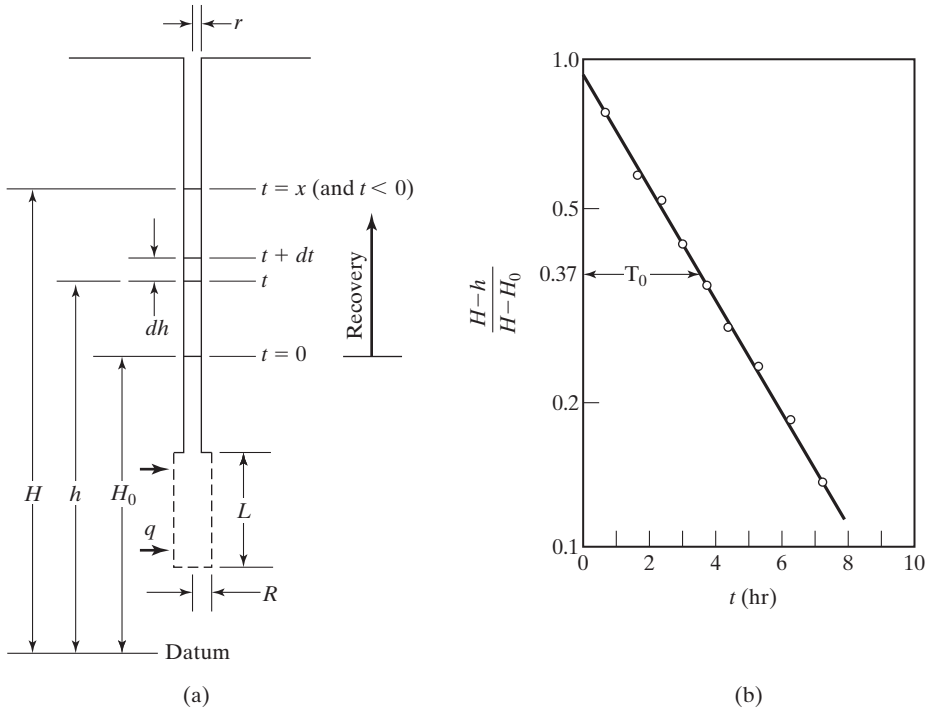
$$T_0 = \frac{\pi r^2}{FK}$$

and solved Equation (8-59) with initial conditions $h = H_0$ at $t = 0$. Thus,

$$\frac{H - h}{H - H_0} = e^{-t/T_0}. \quad (8-60)$$

By plotting recovery $(H - h)/(H - H_0)$ vs. time on semilog graph paper, we find that $t = T_0$, where recovery equals 0.37 (Fig. 8-22). For piezometer intake length divided by radius (L/R) greater than 8, Hvorslev has evaluated the shape factor F and obtained an equation for K :

$$K = \frac{r^2 \ln(L/R)}{2LT_0}. \quad (8-61)$$

**Figure 8-22**

Hvorslev piezometer test. (a) Geometry. (b) Method of analysis.

Several other slug-test methods have been developed by Papadopoulos et al. (1973) for confined aquifers. These methods are similar to Theis's in that a curve-matching procedure is used to obtain S and T for a given aquifer. The solution method is graphical and requires a semilogarithmic plot of measured $H(t)/H_0$ vs. t , where H_0 is the assumed initial excess head. The data are then curve matched to the plotted type curves by horizontal translation until the best match is achieved.

The most commonly used method for determining hydraulic conductivity in ground water investigations is the Bouwer and Rice (1976) slug test. While it was originally designed for unconfined aquifers, it can be used for confined or stratified aquifers if the top of the screen is some distance below the upper confining layer. The method is based on the following equation:

$$K = \frac{r_c^2 \ln(R_e/R)}{2L_e} \frac{1}{t} \ln \frac{y_0}{y_t}, \quad (8-62)$$

where

r_c = radius of casing,

y_0 = vertical difference between water level inside well and water level outside at $t = 0$,

- y_t = vertical difference between water level inside well and water table outside (drawdown) at time t ,
 R_e = effective radial distance over which head is dissipated, and varying with well geometry,
 R = radial distance of undisturbed portion of aquifer from centerline (usually thickness of gravel pack),
 L_e = length of screened, perforated, or otherwise open section of well,
 t = time.

If L_w is less than H in Figure 8-23(a), the saturated thickness of the aquifer, then

$$\ln(R_e/R) = \left[\frac{1.1}{\ln(L_w/R)} + \frac{A + B \ln(H - L_w)/R}{L_e/R_e} \right]^{-1}$$

If L_w is equal to H , then

$$\ln(R_e/R) = \left[\frac{1.1}{\ln(L_w/R)} + \frac{C}{L_e/R_e} \right]^{-1}$$

where A , B , and C are constants that can be found in Figure 8-23(c), plotted as a function of L_e/R .

In Equation (8-62) y_t and t are the only variables. Thus, if a number of y_t and t measurements are taken, they can be plotted on semilogarithmic paper to give a straight line. The slope of the best-fitting straight line will provide a value for $[\ln(y_0/y_t)]/t$. All the other parameters in the above equation are known from well geometry, and K can be calculated. A point to note is that drawdown on the ground water table becomes increasingly significant as the test progresses, and the points will begin to deviate from the straight line for large t and small y . Hence, only the straight-line portion of the data must be used in the calculation for K , as shown in Figure 8-23.

Radial Flow in a Leaky Aquifer

Leaky aquifers represent a unique and complex problem in well mechanics. When a leaky aquifer is pumped, as shown in Figure 8-24, water is withdrawn both from the lower aquifer and from the saturated portion of the overlying aquitard. By creating a lowered piezometric surface below the water table, ground water can migrate vertically downward and then move horizontally to the well. While steady-state conditions in a leaky system are possible, a more general nonequilibrium analysis for unsteady flow is more applicable and more often occurs in the field. When pumping starts from a well in a leaky aquifer, drawdown of the piezometric surface can be given by

$$s' = \frac{Q}{4\pi T} W\left(u, \frac{r}{B}\right), \quad (8-63)$$

where the quantity r/B is given by

$$\frac{r}{B} = \frac{r}{\sqrt{T/(K'/b')}},$$

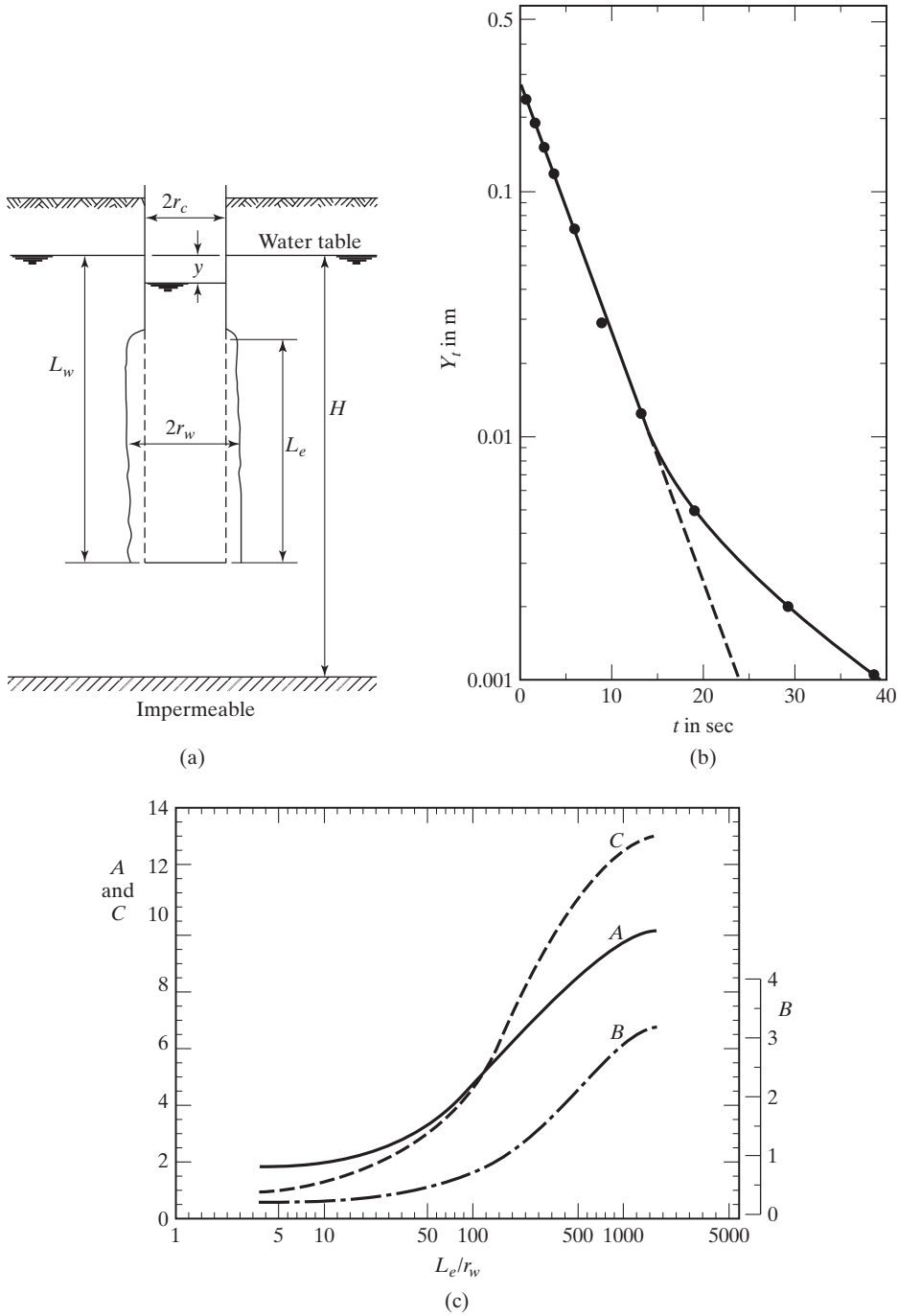
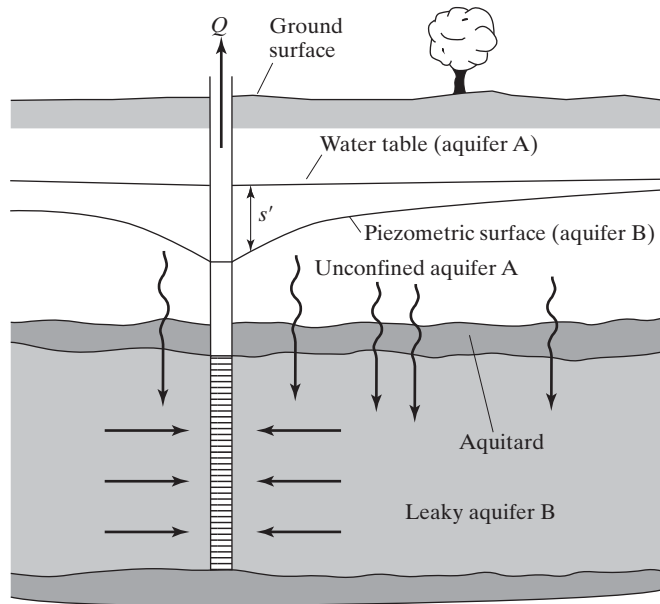


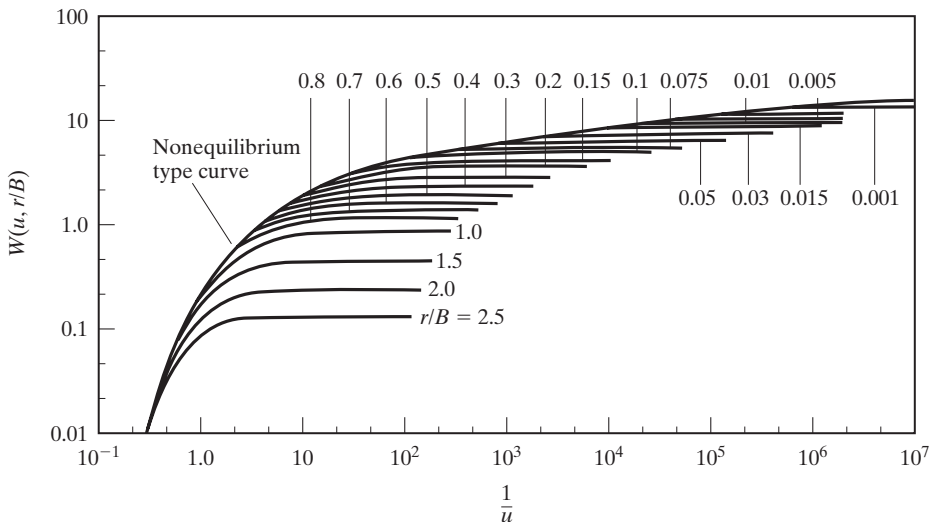
Figure 8-23
Illustration of the Bouwer and Rice slug test.

Figure 8–24

Well pumping from a leaky aquifer.



where T is transmissivity of the aquifer, K' is vertical hydraulic conductivity of the aquitard, and b' is thickness of the aquitard. Values of the function $W(u, r/B)$ have been tabulated by Hantush (1956) and have been used by Walton (1960) to prepare a family of type curves, shown in Figure 8–25. Equation (8–61) reduces to the Theis equation for $K' = 0$ and $r/B = 0$.

**Figure 8–25**

Type curves for analysis of pumping test data to evaluate storage coefficient and transmissivity of leaky aquifers. (After Walton, 1960, Illinois State Water Survey.)

The method of solution for the leaky aquifer works in the same way as the Theis solution with a superposition of drawdown data on top of the leaky type curves. A curve of best fit is selected, and values of W , $1/u$, s' , and t are found, which allows T and S to be determined. Finally, based on the value of r/B , it is possible to calculate K' and b' .

In general, leaky aquifers are much more difficult to deal with than confined or unconfined systems. But the method just described does provide a useful tool for evaluating leaky systems analytically. For more complex geologies and systems with lenses, a three-dimensional computer simulation must be employed to properly represent ground water flow. These types of numerical models and their applications are described in detail in Section 8.12.

Shallow Well Construction

8.11 WATER WELLS

A **water well** is a vertical hole dug into the earth, usually designed for bringing ground water to the surface. Wells can be used for pumping, artificial recharge, waste disposal, water level observation, and water quality monitoring. A variety of methods exist for constructing wells, depending on the flow rate, depth to ground water, geologic condition, casing material, and economic factors.

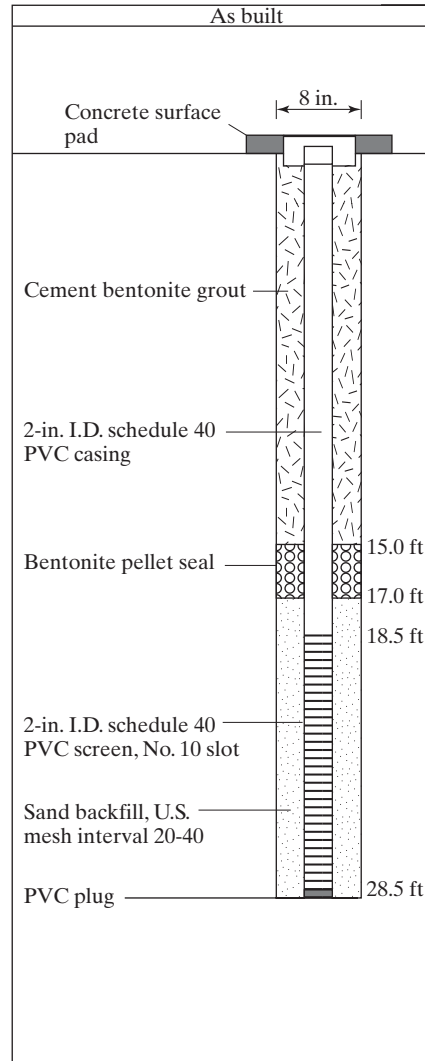
Prior to drilling a well in a new area, a test hole is normally drilled and a record, or log, is kept of various geologic formations and the depths at which they are encountered. Cable tool, rotary, and jetting methods are commonly employed for making test holes. Sample cuttings are often collected at selected depths and later studied and analyzed for grain size distribution. Borings can be removed using a Shelby tube or a split-spoon sampler, and these cores can be analyzed for a variety of parameters, such as hydraulic conductivity, porosity, grain size, texture, and soil classification.

Shallow wells are usually less than 15 m deep and are constructed by digging, boring, driving, or jetting methods. Dug wells are generally excavated by hand and are vertical holes in the ground that intersect the water table. A modern domestic dug well should have a rock curb, a concrete seal, and a pump to lift water to the surface. Bored wells are constructed with hand-operated or power-driven augers, available in several shapes and sizes, with cutting blades that bore into the ground with a rotary motion. When the blades fill with loose earth, the auger is removed from the hole and emptied. Depths up to 30 m can be achieved under favorable conditions. Hand-bored wells seldom exceed 20 cm in diameter.

A continuous-flight power auger has a spiral extending from the bottom of the hole to the surface, and sections of the auger may be added as depth increases. Cuttings are carried to the surface in the drilling process, and depths up to 50 m can be achieved with truck-mounted equipment. Hollow-stem augers are often used to construct small-diameter wells, which are associated with waste sites for monitoring or pumping. Augers work best in formations that do not cave, such as where some clay or silt is present in the aquifer. Figure 8-26 shows a typical example well drilled with a hollow-stem

Figure 8-26

Typical well designs for unconsolidated formations.



auger, where bentonite is used to seal the casing and sand is used near the screen interval.

Driven wells consist of a series of pipe lengths driven vertically downward by repeated impacts into the ground. Water enters the wells through a drive point at the lower end, which consists of a screened, cylindrical section, protected by a steel cone. Driven wells usually have diameters of less than 10 cm, with depths generally below 15 m. Yields from driven wells are usually fairly small, but a battery of such wells connected to a single pipe and pump are effective for lowering water-table elevations. Such a system is known as a well-point system and is used for de-watering excavations for subsurface construction. Jetted wells are constructed with a high-velocity

stream of water directed vertically downward, while the casing that is lowered into the hole conducts the water and cuttings to the surface. Small-diameter holes, up to 10 cm, with depths up to 15 m, can be installed in unconsolidated formations. Because of the speed of installation, jetted wells are useful for observation wells and well-point systems.

Deep Well Construction

Most deep wells of high capacity are constructed by drilling with a cable tool or with one of several rotary methods. Construction techniques vary depending on the type of geologic formation, depth of the well, and required flow rate. Cable tool drilling is accomplished by regular lifting and dropping of a string of tools, with a sharp bit on the lower end to break rock by impact. The method is most useful for consolidated rock materials to depths of 600 m.

A rapid method for drilling in unconsolidated formations (sands or silts) is the rotary method, which consists of drilling with a hollow, rotating bit, with drilling mud, or water used to increase efficiency. No casing is required with drilling mud, because the mud forms a clay lining on the wall of the well.

Drilling mud consists of a suspension of water, bentonite clay, and various organic additives. Proper mixture is essential for trouble-free drilling. This method is used extensively for oil wells. Advantages are the rapid drilling rate and the convenience for electric logging. Air rotary methods use compressed air in place of drilling mud and are convenient for consolidated formations. Drilling depths can exceed 150 m under favorable conditions.

Well Completion and Pumping

Once a well has been drilled, it must be completed in such a way that it remains an efficient producer of water. This may involve installation of casing, cementing, placement of well screens, and gravel packs. Well casings serve as a lining to maintain an open hole up to the ground surface. They provide structural support against caving materials and seal out surface water. Casing materials include wrought iron, steel, and PVC pipe.

Wells are cemented in the annular space around the casing to prevent entrance of water of unsatisfactory quality. Cement grout, which may contain additives, is usually introduced at the bottom of the space to ensure proper sealing. **Grout seals** are common for layered aquifers, where it is desirable to pump from one layer while not disturbing another. **Well screens**, typically used for unconsolidated formations, are perforated sections of pipe of variable length that allow ground water to flow into the well. Their main purpose is to prevent aquifer material, such as sand or gravel, from entering the well and to minimize hydraulic resistance to flow. Screens are made of a variety of metals and metal alloys, PVC, and wood. Where a well screen is to be surrounded by an artificial **gravel pack**, the size of the screen openings is governed by the size of the gravel (see Fig. 8-26).

A gravel pack that envelops a well screen is designed to stabilize the aquifer and provide an annular zone of high-permeability material. In layered systems, gravel packs can provide conduits of flow from one layer to another and should be separated by grout plugs. Details on gravel-pack material and well-screen sizes are available from Campbell and Lehr (1973).

Well development follows completion and is designed to increase the hydraulic efficiency by removing the finer material from the formation surrounding the screen. The importance of well development is often overlooked but is required to produce full-potential yields. Development procedures include pumping, surging, use of compressed air, hydraulic jetting, chemical addition, and use of explosives. The methods are discussed in Campbell and Lehr (1973) and will not be described in detail here.

For shallow wells, low-capacity and low-head pumps can be used. For deeper wells used to supply water for municipal or agricultural needs, deep-well turbine and submersible pumps are needed. Pump impellers are contained in bowls and can be connected in series to deliver high heads. They are suspended vertically on a long drive shaft driven by a surface-mounted motor. Whenever ground water is pumped and is to be used for human consumption, proper sanitary precautions must be taken to protect water quality. Surface pollution can enter wells through the annular space or through the top of the well itself. Cement grout outside the casing and a watertight cover of concrete should be installed for protection. Samples of well water should be evaluated for quality after development and after periods of excessive flooding, if contamination is suspected. Wells are often associated with septic tank systems in rural areas, and most communities have requirements on the distance necessary to separate a water well from the nearest septic tank as a measure of sanitary protection. Ground water contamination in wells is discussed in detail in Freeze and Cherry (1979), Bedient et al. (1999), and Charbeneau (2000).

8.12

GROUND WATER MODELING TECHNIQUES

Use of Ground Water Models

Ground water models have become significant tools of analysis in the past two decades, with increasing use of digital computers and greater access to data from federal and state agencies. Mathematical models in ground water have been greatly expanded and improved in recent years, especially with the advent of graphical user interfaces (GUI). One great advantage of a computer model is its flexibility—it can be used to model a large number of different problems with the same program by simply varying input data such as pumping rates or boundary conditions. Some computer codes are extremely versatile, treating both saturated and unsaturated flow in two or three dimensions. It is beyond the scope of this text to address ground water modeling in detail, but heterogeneous and layered aquifer systems can be modeled easily today, using finite-difference and finite-element numerical techniques. More details and examples are described in Anderson and Woessner (1992), Zheng and Bennett (1995), Bedient et al. (1999), and Charbeneau (2000).

A completely revised and updated version of the original Trescott et al. (1976) flow model from the USGS was released in 1984 as MODFLOW, with a second update in 1988 by the USGS, authored by McDonald and Harbaugh (1988). A recent version of the model was released by Harbaugh et al. (2000) with many additional features. This standard flow code is a modular, three-dimensional finite-difference ground water flow model. The model is designed so that it can be readily modified, can be executed on a variety of platforms, and is relatively efficient with respect to computer memory and execution time. The model's structure consists of a main program and a series of highly independent subroutines called modules. Each module deals with a specific feature of the hydrologic system, such as flow from rivers, flow from wells, or flow into drains. The numerical procedures are included in separate modules and can be selected by the user. The model documentation is extremely complete and easy to use and represents the state of the art in ground water flow models. A number of GUIs have been created to make running the model and dealing with large data sets much easier. MODFLOW is highlighted in the next section for a case study involving aquifer response to overpumping one of the main aquifers in Florida. More details can be found in Bedient et al. (1999) and Todd and Mays (2005).

Particle-tracking computer programs such as MODPATH from Pollock (1988) have become very popular in recent years to predict directions and rates of flow within groundwater systems. Particles are placed at known source areas and move according to the ground water velocity in particular layers of an aquifer. The particles are used to visualize the flow field and track contaminant movement from source to receptor. MODPATH is very useful in evaluating different model parameters and boundary-condition effects. MODPATH is sometimes used in place of a contaminant transport model to predict where contaminants might migrate through time.

Introduction

Ground water overpumping and aquifer depletion are now a serious problem in many parts of the United States and around the world. Aquifers in New York, Texas, Idaho, Arizona, Washington, and California have all felt the impacts of overpumping and overuse (USGS, 1999). Worldwide, the consumption of fresh water is doubling every 20 years, which is more than twice the rate of the increase in human population (Barlow, 1999). This massive consumption of water is placing enormous pressures on areas where aquatic ecosystems dominate, especially lakes and wetlands, as in Florida.

The following is an overview of a case study that concerns ground water pumping in the layered, Karst aquifer system near Tampa, Florida. Impacts of overpumping are discussed, with a brief description of the study area. The

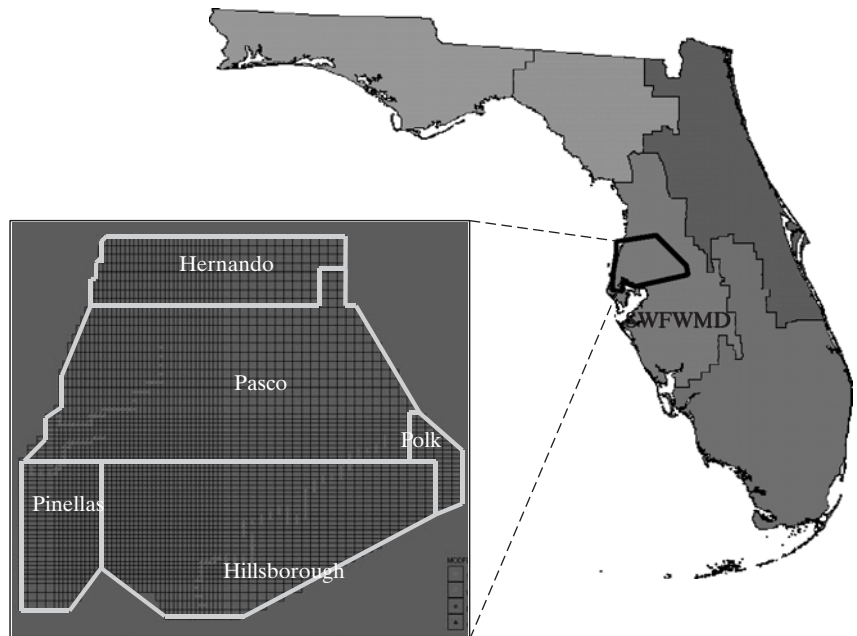
MODELING LAYERED AQUIFER SYSTEMS: A CASE STUDY OF THE EFFECTS OF GROUND WATER OVERPUMPING

Southwest Florida Water Management District (SWFWMD) had an existing MODFLOW model based on data from 1993, and the original model was updated using new information incorporated through 1998. The results of present and projected pumping of ground water were evaluated using existing and improved numerical ground water flow models, based on the original model. The case study provides the student with a unique view of how such models are applied for a large and complex aquifer system, in order to better manage the water resources in the five-county region of southwest Florida.

In Florida, water resources are an integral part of the state's welfare, affecting the state's economy, natural environment, and future growth possibilities. Florida currently relies on ground water for over 90% of its water supply. Florida's increase in population in the last few decades has led to additional development and an intensified water need for agriculture, industry, and public supplies. In order to provide for this additional water use and to plan for future growth and demand, the state has worked on developing long-range plans for protecting its water resources, at both the state and local levels.

One of the state's most stressed areas in terms of water resources is located in the Northern Tampa Bay (NTB) region (Fig. 8-27). Ground water pumping for water supply in the NTB area, which encompasses part or all of Hillsborough, Hernando, Pasco, Pinellas, and Polk counties, has doubled in the past 30 years (Fig. 8-28). In 1966, 44.2 million gallons per day (mgd) of ground water was pumped from the regional wellfields, but by 1996, the

Figure 8-27
Counties encompassed in NTB
wrap area model.
(SWFWMD,
1996.)



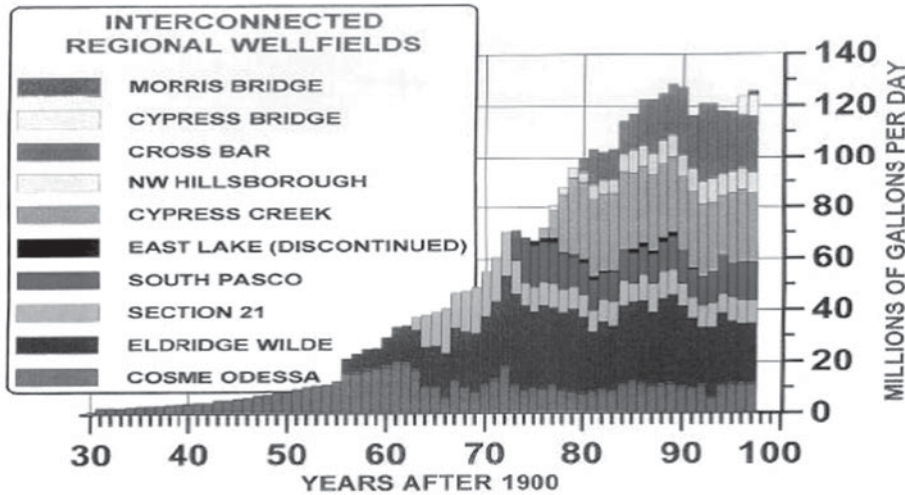


Figure 8–28

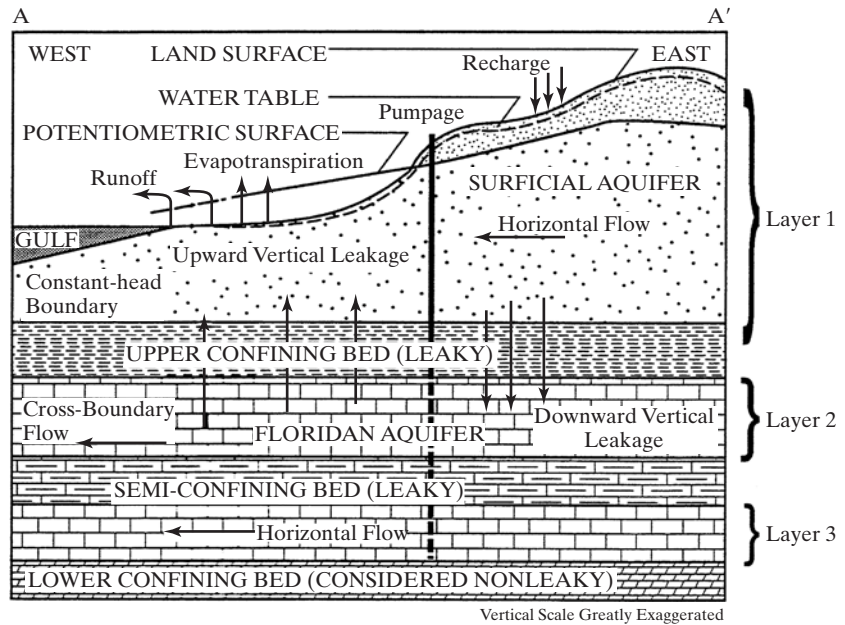
Ground water pumping in NTB area well fields. (SWFWMD, 1999.)

pumping rate was estimated to be 122 mgd (SWFWMD, 1993). Withdrawals have put a large strain on the hydrologic system and have caused serious problems to the region. The need for additional ground water is expected to increase in the future. The greatest increase in ground water use is projected to be in the southern part of the district, which is the same area that already shows tremendous stress on the Floridan Aquifer, the main drinking-water supply aquifer (SWFWMD, 1999). District studies have projected that if current trends continue, ground water pumping in NTB counties will have increased significantly by 2035.

Aquifer Systems and Overpumping

In order to quantify the ground water effects of pumping in a complex system, it is important to first identify the interactions between the ground water and the surface water. Ground water and surface water are intricately interconnected in the NTB area because of the Karst limestone features that underlie the area. Figure 8–29 shows many of the hydraulic links that often exist in Karst systems, such as fractures and layering of units. Water enters the ground water system through pathways such as rainfall and recharge, and it leaves the system through pathways such as runoff, ground water outflow, and evapotranspiration. Many interactions are also occurring between the different layers of the system. For example, the NTB area has two area aquifers, the surficial aquifer and the Floridan Aquifer, which are separated by leaky confining beds. Vertical transfer of water occurs across these leaky layers and is related to rates of pumping from the lower aquifer (Figs. 8–24 and 8–29).

Figure 8–29
Surface and ground water interactions.
(Modified from Hutchinson, 1984.)



The NTB MODFLOW model area covers approximately 1500 square miles and includes two principal aquifers, the surficial aquifer and the Upper Floridan Aquifer. The NTB model represents these aquifers as three layers, with layer 1 simulating the surficial aquifer, layer 2 simulating the Tampa/Suwanee portion of the Floridan Aquifer, and layer 3 simulating the Avon Park Formation portion of the Floridan Aquifer. The surficial aquifer is defined to be approximately 35 ft thick, while the Upper Floridan is a total of 900 ft thick and is divided into 500 ft in layer 2 and 400 ft in layer 3. The aquifer layers are separated by semiconfining zones, which allow for vertical interchange between the layers (USGS, 1989).

Due to the Karst geology present in this area, the area's ground water and surface water are closely interconnected, which leads to major concerns over the present and projected increases in pumping throughout the area. Overpumping of water from aquifer systems can cause both direct and indirect problems. Direct effects include lowering of the potentiometric surface of the aquifer, lowering the water table of the shallow aquifer, lowering of the amount of water available for surface runoff, deterioration of water quality, and increased pumping costs (Drew and Hotzl, 1999). An example of water quality deterioration is the potential for seawater intrusion. Overpumping of coastal aquifers produces a lowering of water-table levels, which are protected by the recharge in the coastal strip of the sea level. When extracted volumes are greater than the recharge, saltwater begins to intrude into the aquifer (Freeze and Cherry, 1979). In the Pinellas County coastal subregion, a study done by the SWFWMD detailed that water quality trends indicate localized seawater intrusion may be occurring. Chloride data from wells in this area show the number of significant

increasing chloride trends (SWFWMD, 1999). For the interested student, a more detailed discussion of saltwater intrusion can be found in Fetter (1994).

One of the most serious effects observed in the NTB area has been the decline of ground water levels, which have dropped in some areas as much as 20 ft between the 1940s and 1990s (SWFWMD, 1993). The decline in aquifer water levels can also cause many problems with surface water bodies. In the NTB area there is currently concern over the effect of lowered water levels on lakes, wetlands, springs, and reduced stream flow in river systems caused by overpumping in the area. The effect of pumping on the surface water bodies is dependent on a variety of factors. These include

- the distance between the pumping well and the lake,
- the rate and duration of pumping,
- geologic and hydrologic characteristics of the aquifer,
- vertical permeability and thickness of the confining beds,
- difference in head established between the deeper aquifer and the shallow water-table aquifer, and
- the degree of interconnection between the lake bottoms and the underlying unconsolidated sediments and the limestone (Stewart, 1968).

Since there are hundreds of lakes and wetlands dispersed throughout the NTB area, many of which are near the major pumping centers, the area's surface water bodies have been greatly affected (SWFWMD, 1993).

Data Collection and Model Input

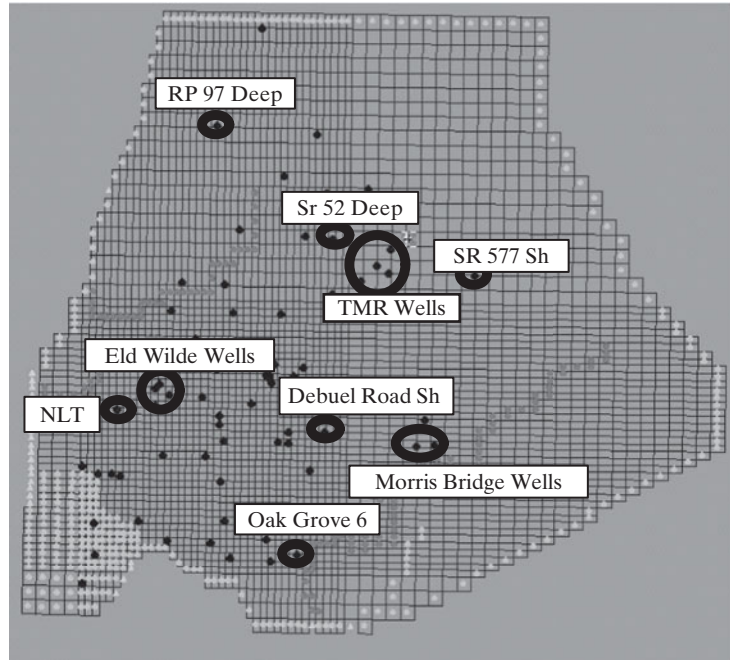
The NTB model is an example of a regional model that includes 11 well fields in a five-county region that covers a total of 1500 square miles (SWFWMD, 1993). The NTB model takes advantage of the flexibility allowed by MODFLOW in having variable spacing with grid-cell areas ranging from 0.25 to 1 square mile. This variable spacing was selected to allow for smaller grid cells around areas of high ground water use and areas with well-defined hydrogeological information. The grid area includes 62 rows and 69 columns that are bounded by constant-head boundaries, no-flow boundaries, and general head boundaries.

Observation data were used to assess both spatial and temporal agreement of the models. For temporal comparisons, all available data points were compared; but for spatial comparisons, 17 locations were selected as representative samples across the region (Fig. 8–30). These locations were selected to be representative of both aquifer levels as well as to be widely distributed across the grid. The three clusters of wells near Eldridge Wilde and Morris Bridge well fields were selected to study the model's ability to simulate the local impacts around large-volume pumping centers.

Ground water models require input information to define the hydrogeologic and hydrogeochemical conditions of the area. The input information is categorized into three areas: geologic, physical and chemical parameters, and

Figure 8–30

Spatial comparisons of observation station locations. Seventeen locations were selected to be representative of both aquifer levels as well as to be widely distributed across the grid.



external stress parameters. The geologic inputs outline the location of the boundaries and thickness of geologic units in the model. The external stresses include inputs on pumping, injection, and recharge. The physical and chemical parameter inputs define the hydraulic conductivity, porosity, transmissivity, and evapotranspiration of each grid cell. The recharge value is based on land use, average rainfall in subbasins, and a recharge coefficient predetermined by the water district. Changes in growth and development require temporal updating of parameters such as pumping, recharge, and evapotranspiration. It is important to note that model input parameters are never completely defined; there is always associated uncertainty of sampling and interpolation error (Zheng and Bennett, 1995). The calibrated SWFWMD 1993 model was used as the starting point for this study. The calibrated model provided all initial estimates of transmissivity, hydraulic conductivity, storage, leakage, recharge, and evapotranspiration. These values are based on model data sets collected from the tricounty well-field model and the Pasco County water resources model (Fretwell, 1988) and on field data collected by the NTB project and other investigators.

GMS and Modflow

The Groundwater Modeling System (GMS) model is a robust and comprehensive ground water modeling package that provides tools for every phase of a ground water simulation, including site description, model development, postprocessing, calibration, and visualization. Modeling with GMS allows

the user variation and flexibility in graphical output. GMS includes a sophisticated multilayer MODFLOW interface that allows models to be defined and edited at the conceptual model level or on a cell-by-cell basis at the grid level. MODFLOW is one of the most widely used flow models and consists of a main program and a series of independent subroutines grouped into modules, or packages. Each package deals with a specific feature of the hydrologic system, such as flow from rivers, recharge, or transmissivity. The division of the program into packages allows the user to analyze the specific hydrologic features of the model independently.

MODFLOW is a three-dimensional finite-difference ground water flow model, developed for the USGS by McDonald and Harbaugh (1988). MODFLOW is based on solving the following partial differential equation for head in three-dimensional movement of ground water of constant density through porous media:

$$\frac{\partial}{\partial x} \left(K_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_{yy} \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left(K_{zz} \frac{\partial h}{\partial z} \right) - W = S_s \frac{\partial h}{\partial t}, \quad (8-64)$$

where K_{xx} , K_{yy} , and K_{zz} are defined as the hydraulic conductivity along the coordinate axis, h represents the potentiometric head, W is the volumetric flux per unit volume being pumped, S_s is the specific storage of the aquifer, and t is the time.

MODFLOW uses a block-centered finite-difference grid approach in which flux boundaries are always located at the edge of the block (Anderson and Woessner, 1992). The model can handle unconfined or confined systems, external stresses such as wells, areal recharge, evapotranspiration, flow to drains, and flow through rivers. It is considered the standard flow model in the field and is often linked with other contaminant transport models to simulate movement of chemicals in the subsurface. The model is designed so that it can be modified easily and executed on a variety of computers, and it is efficient to use with graphical programs such as GMS (Bedient et al., 1999).

Case Study Results

In order to update the SWFWMD's model, which was based upon data from 1993, the MODFLOW model needed to be restructured using data through 1998. Simulations were performed using various combinations of pumping, recharge, transmissivity, and evapotranspiration input files. Results were analyzed for both temporal and spatial agreement. Recharge was a main parameter not known with significant accuracy due to lack of necessary hydrogeologic data, changes in land use, rainfall pattern variation, and the fact that some overprediction was observed in the original 1993 model runs. In addition, recharge was evaluated by the SWFWMD on an annual basis, which masked fluctuations that occurred on a monthly basis. Thus, recharge was altered to account for the variability in data; several recharge data sets were considered, which utilized the current annual average recharge from

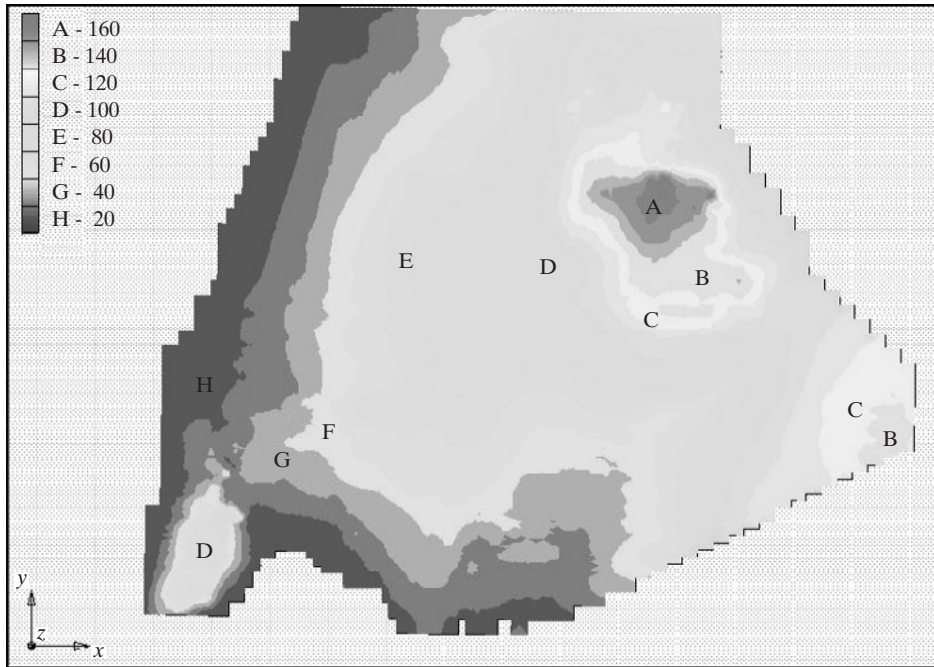


Figure 8-31

Contour of simulated heads (ft) for December 1998, using 70% recharge simulation.

the district as well as reduced values of 90% recharge, 80% recharge, 75% recharge, and 70% recharge. For example, the 70% recharge implies that the original district recharge values have been multiplied by 70%.

Qualitative analysis was done using graphical features within GMS for MODFLOW, including comparing contour plots of heads and drawdowns and plots of observed versus simulated values. Figure 8-31 shows typical results of the analysis from GMS, displaying the contours of simulated heads for December 1998, using the 70% recharge simulation. This analysis with GMS was useful in initial model evaluation and development; however, in order to identify the simulation that best represented the observed conditions, quantitative statistical analyses were needed.

Model accuracy was improved for the existing NTB model by using monthly variations and improved inputs for 1989–1998. These improvements were evaluated using comparative statistical analysis. In order to have confidence in the results of this study, it was first necessary to demonstrate that the model was simulating the area within reasonable error and that the error was improved over previous studies. Both temporal and spatial statistical comparisons, using 104 observation stations and 40 representative time steps, were performed for the period from 1989–1998. The 70% recharge simulation, as shown in Figure 8-31, proved to be the most accurate simulation tested, with a mean absolute error (MAE) of 4.34 ft, a mean error (ME) of 0.24 ft, and an r^2 correlation of 0.935.

ME is computed as the difference between the measured head and simulated heads; values closer to zero indicate the best overall agreement. MAE is the absolute value of the difference between the measured head and simulated head. The r^2 correlation indicates the scatter between the simulated and observed data. These errors are all smaller than the district's previous model, which had an MAE of 6.48 ft and consistently overpredicted with an average ME of -5.06 ft.

Future Conditions

Future conditions were projected with the model with current pumping rates from the Upper Floridan Aquifer (2010 Current Pumpage) along with the 70% recharge condition. The 2010 Current Pumpage simulation results indicated water levels in the aquifer would remain on average within 0.1 ft of the 1999 water levels that were estimated by the district (SWFMWD, 1993). However, both the 1999 and the 2010 water levels averaged 6.8 ft lower than historic levels. Since the predicted levels were generally much lower than historic levels, continued damage to lakes, wetlands, and the aquifers is expected. Thus, some reduction in pumpage is clearly needed.

Future conditions were also projected with reduced pumping rates in the aquifer. When testing the effect of reduced pumping using the 2010 Reduced Pumpage simulation, the model results indicated water levels increasing at the stations located near pumping fields. These 2010 water levels had an average increase of 3.25 ft from estimated 1999 water levels, were much closer to historic levels than the current projections, and were on an average 3.4 ft lower than historic levels. Although some locations did not meet the historic levels, they did improve on the current conditions, which should decrease the magnitude of impacts.

Figure 8–32 presents results from the SR 52 Deep Recovery Station and shows the original district prediction, the district 70% recharge, 2010 Current Pumpage, and 2010 Reduced Pumpage results. All four computer runs were compared against the Required Recovery Level, the level where further withdrawal would be significantly harmful to the water resources of

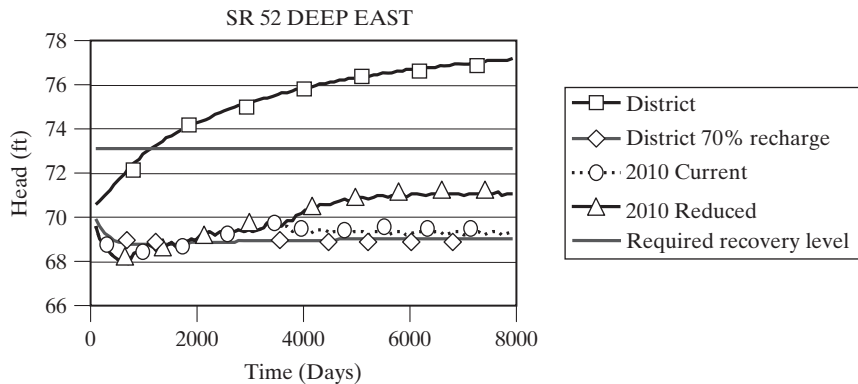


Figure 8–32
Simulation results
for SR 52 Deep
East Recovery
Station.

the area. Continued work with the model is needed to better calibrate the past and the future conditions, as more data become available through time.

The use of GMS and MODFLOW enabled predictions using many different variations of the governing ground water parameters. Input and output data sets were efficiently managed with the GMS software and allowed a complex modeling exercise in MODFLOW to be handled with relative ease. Several other excellent case studies on ground water modeling can be found in the literature and in Anderson and Woessner (1992), Bedient et al. (1999), Charbeneau (2000), and Todd and Mays (2000).

SUMMARY Ground water hydrology is introduced and reviewed in this chapter. Ground water flow according to Darcy's law is described, and the general flow equations are derived for confined and unconfined aquifers. The theory of streamlines and equipotential lines is presented to explain flow-net concepts. Dupuit flow and seepage are shown for unconfined aquifers along with equations for steady radial flow to a well for single-well and multiple-well systems. The concept of superposition is presented and can be used for handling complex flow fields.

Theis's method for unsteady well hydraulics is derived in theory and several applications are included; it forms the modern basis for aquifer pump tests. The simpler Cooper-Jacob method can be used to solve the unsteady-flow equation under certain conditions. Finally, radial flow in a leaky aquifer is presented in theory and example. Slug tests are used to determine aquifer properties using a single well, whereas pump tests require more than one well.

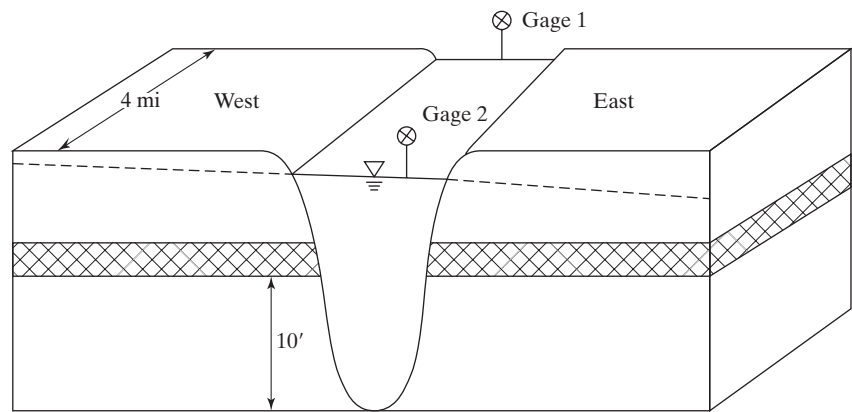
Ground water modeling techniques include both analytical and numerical solutions to the governing flow equations. Usually, finite-difference methods for steady-state or transient flow are used, especially where heterogeneous properties exist. One of the most popular computer models for ground water flow is MODFLOW. It has become the model of choice for any ground water study of regional hydrogeology or contaminant transport. When used along with MODPATH to predict flow directions and streamlines, it can represent any layered aquifer system along with complex boundary and initial conditions for a site. A case study using MODFLOW applied to an aquifer in Florida is described in detail to provide a useful level of coverage for the beginning student or professional. Ground water modeling case studies can be found in the literature and in Anderson and Woessner (1992), Bedient et al. (1999), Charbeneau (2000), and Todd and Mays (2000).

PROBLEMS **8.1.** Compute the Darcy velocity and seepage velocity for water flowing through a sand column with the following characteristics:

$$\begin{aligned}K &= 10^{-4} \text{ cm/s,} \\dh/dl &= 0.01, \\ \text{area} &= 75 \text{ cm}^2, \\n &= 0.30.\end{aligned}$$

- 8.2.** The average water-table elevation has dropped 4 ft due to the removal of 100,000 ac-ft from an unconfined aquifer over an area of 75 mi². Determine the storage coefficient for the aquifer.
- 8.3.** A confined aquifer is 50 m thick and 0.5 km wide. Two observation wells are located 1.5 km apart in the direction of flow. Head in well 1 is 50.0 m and in well 2 is 42 m. Hydraulic conductivity K is 0.7 m/day.
- (a) What is the total daily flow of water through the aquifer?
 - (b) What is the height z of the piezometric surface 0.5 km from well 1 and 0.9 km from well 2?
- 8.4.** A well with a diameter of 18 in. penetrates an unconfined aquifer that is 100 ft thick. Two observation wells are located at 100 ft and 235 ft from the well, and the measured drawdowns are 22.2 ft and 21 ft, respectively. Flow is steady and the hydraulic conductivity is 1400 gpd/ft². What is the steady-state rate of discharge from the well?
- 8.5.** Two piezometers are located 1200 ft apart with the bottom located at depths of 50 ft and 350 ft, respectively, in a 400-ft-thick unconfined aquifer. The depth to the water table is 50 ft in the deeper piezometer and 40 ft in the shallow one. Assume that hydraulic conductivity is 0.0002 ft/s.
- (a) Use the Dupuit equation to calculate the height of the water table midway between the piezometers.
 - (b) Find the flow rate per unit thickness for a section midway between the wells.
- 8.6.** In a fully penetrating well, the equilibrium drawdown is 30 ft measured at $r = 100$ ft from the well, which pumps at a rate of 20 gpm. The aquifer is unconfined with $K = 30$ ft/day, and the saturated thickness is 100 ft. What is the steady-state drawdown at the well ($r = 0.5$ ft) for this aquifer?
- 8.7.** A soil sample 6 in. in diameter and 1 ft long is placed in a falling-head permeameter. The falling-head tube diameter is 1 in. and the initial head is 8 in. The head falls 1 in. over a 2-hr period. Calculate the hydraulic conductivity.
- 8.8.** A constant-head permeameter containing very fine-grained sand has a length of 12 cm and a cross-sectional area of 30 cm². With a head of 10 cm, a total of 100 ml of water is collected in 30 min. Find the hydraulic conductivity.
- 8.9.** Bull Creek flows through and completely penetrates a confined aquifer 10 ft thick, as shown in Figure P8-9. The flow is reduced in the stream by 16 cfs between two gaging stations located 4 miles apart along the creek. On the west side of the creek, the piezometric contours parallel the bank and slope toward Bull Creek at $S = 0.0004$ ft/ft. The piezometric contours on the east side of the creek slope away from the channel at a slope of 0.0006 ft/ft. Using Darcy's law and the continuity equation, compute the transmissivity of the aquifer along this section of Bull Creek. Note that the flow area to the creek is 15 ft thick and 4 miles wide.

Figure P8–9



8.10. Three geologic formations overlie one another with the characteristics listed below. A constant-velocity vertical flow field exists across the three formations. The hydraulic head is 75 ft at the top of the formations and 59 ft at the bottom, with a datum located at the bottom of the three units. Calculate the hydraulic head at the two internal boundaries.

$$\begin{aligned} b_1 &= 20 \text{ ft}, & K_1 &= 20 \text{ ft/day}, \\ b_2 &= 10 \text{ ft}, & K_2 &= 0.20 \text{ ft/day}, \\ b_3 &= 25 \text{ ft}, & K_3 &= 0.030 \text{ ft/day}. \end{aligned}$$

8.11. Two wells are located 100 m apart in a confined aquifer with a transmissivity $T = 2 \cdot 10^{-4} \text{ m}^2/\text{s}$ and storativity $S = 7 \cdot 10^{-5}$. One well to the west is pumped at a rate of $6.6 \text{ m}^3/\text{hr}$ and the other to the east at a rate of $10.0 \text{ m}^3/\text{hr}$. Plot draw-down as a function of distance along the line joining the wells at 1 hr after the pumping starts.

8.12. At a waste site, a Hvorslev slug test was performed in a confined aquifer with a piezometer intake length of 20 ft and a radius of 1 in. The radius of the rod was 0.68 in. The following recovery data for the well were observed. Given that the static water level is 7.58 ft and $H_0 = 6.88 \text{ ft}$, calculate the hydraulic conductivity.

Time (s)	20	45	75	101	138	164	199
h (ft)	6.94	7.00	7.21	7.27	7.34	7.38	7.40

8.13. A well casing with a radius of 2 in. is installed through a confining layer into a formation with a thickness of 10 ft. A screen with a radius of 2 in. is installed in the casing. A slug of water is injected, raising the water level by 0.5 ft initially. Given the following recorded data for head decline, find the values of T and K for this aquifer, using Hvorslev analysis. Assume $H_0 = 0.0$ and $H = 0.5 \text{ ft}$ after injection. Assume $L = 15 \text{ ft}$.

Time (s)	5	10	30	50	80	100	120
h (ft)	0.47	0.42	0.32	0.26	0.15	0.12	0.10

- 8.14.** A small municipal well was pumped for 2 hr at a rate of 15.75 liters/s (0.556 cfs). An observation well was located 50 ft from the pumping well and the following data were recorded. Using the Theis method outlined in Example 8–6, compute T and S .

t (min)	0	1	2	3	5	7	9
s' (ft)	0	1.5	4.0	6.2	8.5	10.0	12.0
t (min)	12	15	20	40	60	90	120
s' (ft)	13.7	14.9	17.0	21.7	23.1	26.0	28.0

- 8.15.** A well in a confined aquifer is pumped at a rate of 833 liters/min (1199.5 m³/day) for a period of over 8 hr. Time–drawdown data for an observation well located 250 m away are given below. The aquifer is 5 m thick. Use the Cooper–Jacob method to find values of T , K , and S for this aquifer.

Time (hr)	Drawdown (m)
0.050	0.091
0.083	0.214
0.133	0.397
0.200	0.640
0.333	0.976
0.400	1.100
0.500	1.250
0.630	1.430
0.780	1.560
0.830	1.620
1.000	1.740
1.170	1.860
1.333	1.920
1.500	2.040
1.670	2.130
2.170	2.290
2.670	2.530
3.333	2.590
4.333	2.810
5.333	2.960
6.333	3.110
8.333	3.320

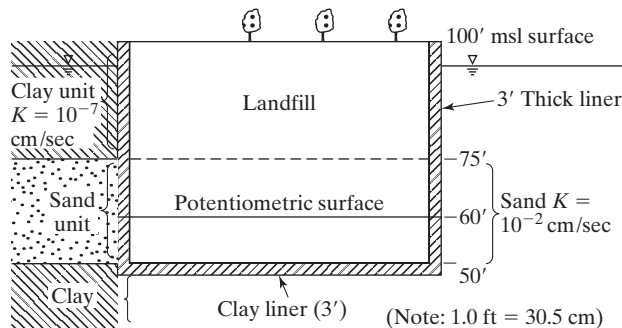
- 8.16.** Drawdown was observed in a well located 100 ft from a pumping well that was pumped at a rate of 1.11 cfs (498 gpm) for a 30-hr period. Use the Cooper–Jacob method to compute T and S for this aquifer.

Time (hr)	1	2	3	4	5	6	8	10	12	18	24	30
Drawdown (ft)	0.6	1.4	2.4	2.9	3.3	4.0	5.2	6.5	7.5	9.1	10.5	11.5

- 8.17.** Repeat Problem 8.16 using the Theis method.
- 8.18.** Refer to Figure 8–9(b), which shows a flow net under a dam section. Note the values of head for the two sides of the dam, and compute the seepage or flow rate through the dam if the dam is 120 ft long with $K = 20$ ft/day.
- 8.19.** A landfill liner is laid at elevation 50 ft msl (mean sea level) on top of a good clay unit. A clean sand unit extends from elevation 50 ft to elevation 75 ft, and another clay unit extends up to the surface, located at elevation 100 ft. The landfill can be represented by a square with length of 500 ft on a side and a vertical depth of 50 ft from the surface. The landfill has a 3-ft-thick clay liner with $K = 10^{-7}$ cm/s around the sides and bottom, as shown in Figure P8–19. The regional ground water level for the confined sand ($K = 10^{-2}$ cm/s) is located at a depth of 10 ft below the surface, or elevation = 90 ft.

How much water will have to be continuously pumped from the landfill to keep the potentiometric surface at 60 ft elevation (msl) within the landfill? Assume mostly horizontal flow through the clay liner.

Figure P8–19



- 8.20.** Repeat Problem 8.19 if the clean sand unit extends below the landfill to an elevation of 25 ft msl. The clay liner exists around and on the bottom of the landfill. Consider Darcy's law across both the sides and bottom of the clay liner.
- 8.21.** A well located at $x = 0, y = 0$ injects water into an aquifer at $Q = 1.0$ cfs, and observation wells are located along the x -axis at $x = 10, 50, 150,$ and 300 ft away from the injection well. The confined aquifer with thickness of 10 ft has $T = 3200$ sq ft/day and $S = 0.005$. The injection is affected by the presence of a linear river located at $x = 300$ ft east of the injection well. Compute the head buildup along the x -axis at the observation-well locations after 6 hr of injection.
- 8.22.** Repeat Dupuit Example 8–3 for the case where net recharge $W = 10$ cm/yr. Repeat the example for the case where $W = 0$.

- 8.23.** Use the Theis-method equations to characterize the behavior of a confined aquifer that is homogeneous and isotropic with $T = 500 \text{ m}^2/\text{day}$ and $S = 1 \times 10^{-5}$. A single well is pumped at $2500 \text{ m}^3/\text{day}$.
- Compute the drawdown 75 m away from the well at $t = 1, 10, 100, 1000$, and $10,000$ minutes after pumping began.
 - Compute the drawdown after 1.0 day of pumping at locations $r = 2, 5, 10, 50, 200$, and 1000 m from the pumping well.
- 8.24.** A well at a distance d from an impermeable boundary pumps at a flow rate Q . The head at any point (x, y) is given by the following equation:

$$h(x, y) = \frac{Q}{2\pi T} [\ln(r_1 r_2)] + C,$$

where C is a constant, r_1 is the straight-line distance from the well to the point (x, y) , and r_2 is the distance from the image well to the point (x, y) . The y -axis lies along the impermeable boundary. Use Darcy's law to show that the flow across the boundary (y -axis) is indeed zero.

- 8.25.** A fully penetrating well pumps from a confined aquifer of thickness 20 m and $K = 10 \text{ m/day}$. The radius of the well is 0.25 m and the recorded pump rate in the well is $100 \text{ m}^3/\text{day}$ at steady state. Assume that the radius of influence of the well is 1250 m. Compute the drawdown at the well if it is located 100 m from an impermeable boundary (see Problem 8.24).

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Chapter 9

Design Applications in Hydrology

—By James F. Thompson



Open channel improvements in Houston, Texas.

9.1 INTRODUCTION This chapter addresses many fundamental design applications in hydrology that are routinely used in engineering and related analyses. The basic design elements of drainage collection systems for urbanized areas are presented along with the applications of synthetic design storms used in common design and analysis situations. Culvert design is presented at a fundamental level, and applications of detention basins for peak runoff control are also discussed as applied to typical urban developments and improvement projects. Design issues applied in floodplain management for the purposes of flood damage reduction are discussed, along with an example of various stages of applications.

Detailed examples of design applications are presented in this chapter, and much of the discussion builds upon other topics in hydrology presented in other chapters. Design approaches commonly used in engineering applications are described, employing conventional hand-calculation methods as well as computer modeling using HEC-HMS, HEC-RAS, and other software programs.

**9.2
DRAINAGE
COLLECTION
SYSTEMS**

Typically, a drainage design is initiated based upon a need either for facilitating drainage of a particular project area or for improved drainage as would be required to address the flood reduction of a particular area. Five steps are normally taken in the design of drainage collection systems:

1. Define the drainage areas serving the project area.
2. Determine the hydrologic response and runoff from these drainage areas, given a particular design or actual storm event.
3. Design the stormwater conveyance conduits.
4. Analyze the effects of increased stormwater quantity, if any.
5. Mitigate any project negative effects.

Another component commonly included in design relates to stormwater quality. Many references address the issue of the management of runoff quality. Local or regional jurisdictions usually control the design parameters and criteria to be employed for a particular collection system.

A typical urban stormwater system consists of streets constructed with curbs, gutters, inlets, and roadside ditches; underground storm sewers; and open outfall channels such as streams, bayous, and rivers receiving runoff. In designing a system, the first basic step is to determine the contributing drainage areas to the system, so that the runoff may be estimated. This entails the development of a drainage area map based upon the topography of the project area. Figure 9–1 illustrates a typical urban setting, upon which a drainage area map has been developed to define the contributing drainage areas to storm sewer system junctions.

More explicit maps are often prepared delineating the individual drainage areas based upon inlet placement. The drainage area map illustrates a simplified layout to be used for design of the storm sewer system. Inlet design and placement are critical and are discussed later in this chapter. In Figure 9–1, each individual drainage area drains to the associated manhole junction (drainage area A9 to manhole junction MH-A9, and so on).

The development of a proper drainage area map is a fundamental building block of any urban stormwater system. Existing topographic maps and other map sources illustrating relative topographic contours may be used to develop the map. More advanced methods may be employed, such as aerial photography, which is in turn used to develop a digital elevation model (DEM). Further developments in remote sensing techniques are being applied today, such as light detection and ranging (LIDAR), which is used to produce very accurate three-dimensional digital maps for use in the development of drainage area boundaries. Regardless of the source, it is imperative that the drainage area map reflect the true pattern of the surface flow of rainfall runoff. Once these patterns and contributing drainage areas are known, the quantity of storm water runoff may be determined. Chapter 10 presents more details on topographic analysis using geographical information systems (GIS).

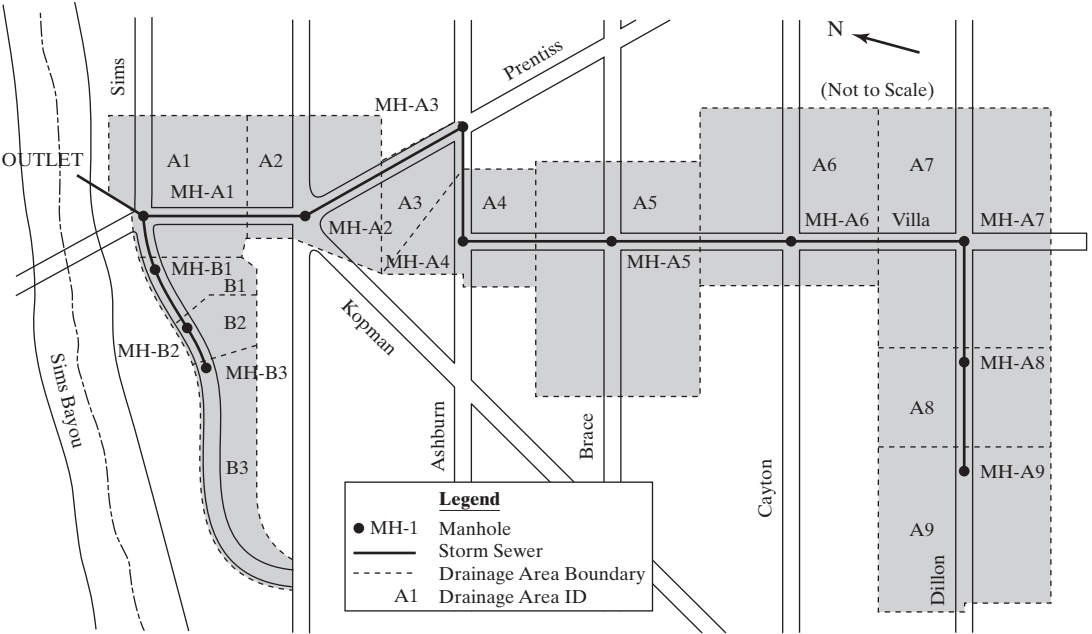


Figure 9-1
Typical drainage area map for an urban project.

Synthetic Design Storms

Synthetic design storms, as explained in Chapter 6, are commonly used in design applications to size particular collection elements such as inlets, conduits such as storm sewers and open ditches or channels, and storage elements in the form of detention basins. The basic principles of establishing synthetic storms entails the creation of a particular storm event hyetograph, as shown in Figure 9-2, relating the intensity (inches per hour) to storm

Figure 9-2
Typical storm
hyetograph.

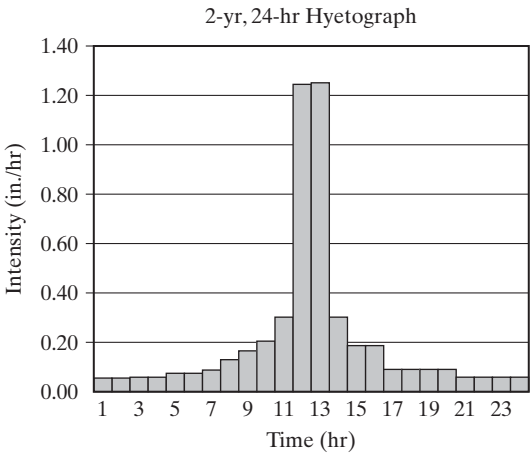


Table 9–1. Design Event Frequencies and Probability of Occurrence

Design Event Frequency	Percent Probability of Occurrence in Any Given Year	Common Applications in Design
2	50%	Local roads and minor channels
5	20%	Local roads and minor channels
10	10%	Principal roads and channels
25	4%	Principal roads and channels
50	2%	Major highways and channels
100	1%	Critical infrastructure and channels
500	0.2%	Critical infrastructure and channels

duration (hours). Particular synthetic storm events are typically associated with a particular return frequency, as shown in Table 9–1. This table also lists common applications for the various synthetic design storms.

Collection systems are designed to convey a particular design storm for a given return period as typically specified by a local governing agency. Some entities (i.e., city, county, and state agencies) require more stringent standard design capacities for local collection systems than do others. The variety of design events utilized for various applications is extensive. In some locations, a 2-yr design storm is suitable for local collector systems, whereas in others, established criteria may require that a larger design storm event be managed by the collection system, such as a 10-yr design storm. For special stormwater management features such as pump stations, even greater design storm events may be required. The particular requirements in terms of collection and conveyance are commonly a function of occurrence, cost, and the associated benefit derived from lower stormwater levels in channels, ditches, streets, and other related infrastructure features.

Rational Method as Applied to Design

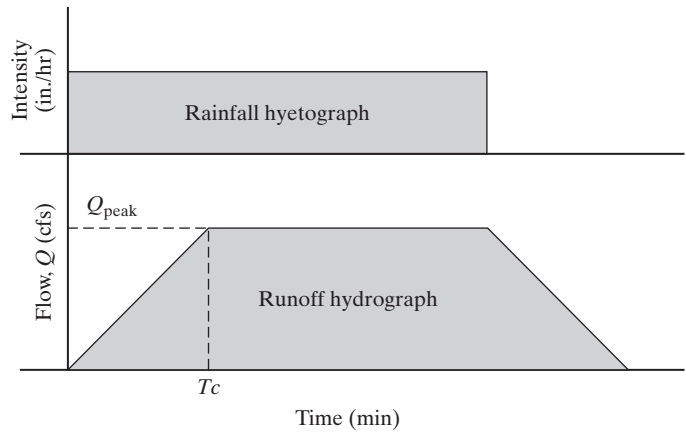
The most commonly used method for determining peak flow rates from a given project drainage area is the rational method using the formula

$$Q = (CA) \cdot i \quad (9-1)$$

where Q is the peak runoff flow in cfs, C is the dimensionless runoff coefficient, A is the drainage area in acres, and i is the rainfall intensity in inches per hour. The rational method is simplistic in that it relies on the basic physical characteristic of a drainage area (i.e., size and runoff coefficient) multiplied by a rainfall intensity as based upon the design event being considered. The runoff coefficient, C , varies dramatically based upon the developed state of the drainage area. Table 6–6 lists commonly utilized runoff coefficients. Section 6.2 presents more details on the rational method.

The rainfall intensity is a function of the return frequency or design event and the time of concentration (T_c). T_c is normally thought of as the equilibrium time for rainfall at the most remote part of the drainage area to contribute flow at the outlet or design point—in other words, how many minutes it takes for a water particle to travel the longest distance within a contributing drainage area

Figure 9–3
Rational formula
hyetograph and
runoff hydrograph.



being considered. The basic fundamental assumption of T_c in the rational method is that the time to peak, T_p , is equal to T_c and that the runoff hydrograph for a project area has a simplified relationship, as shown in Figure 9–3.

A critical function in the rational formula is the rainfall intensity, i . While specific rainfall intensities can be used for a given project area, it is common to use established intensity–duration–frequency (IDF) curves. Using sources relating total rainfall depth to a rainfall duration period (i.e., 5- to 60-minute; and 6-, 12-, and 24-hour, and so on), such as Technical Paper No. 40 and Technical Memorandum Hydro 35 from the National Weather Service, we may develop IDF curves which relate T_c , the design event or frequency, and the runoff intensity measured in inches per hour. Most major municipalities and state entities have IDF curves, or formulas used to create specific IDF curves, readily available for use. Furthermore, these same entities typically specify the design event to be utilized on a given project. Typical IDF curves are illustrated in Figure 9–4.

Figure 9–4
Typical IDF
curves.

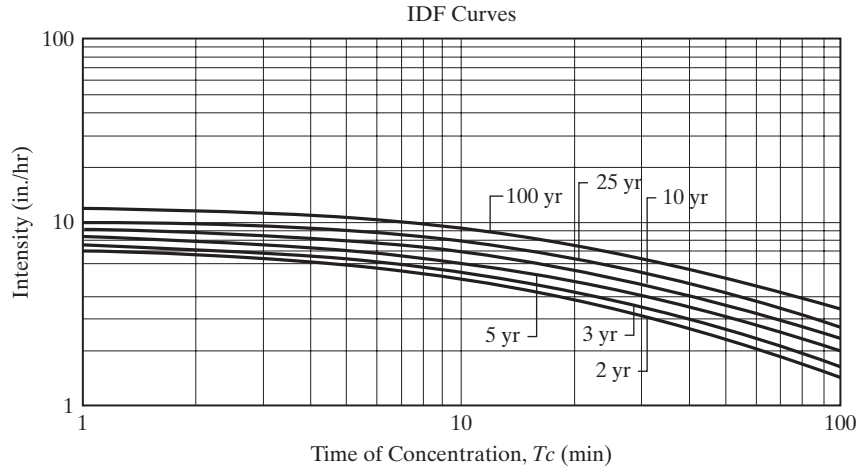


Table 9–2. Coefficients for Rational Method Intensity Formula

	Design Storm					
	2 yr	3 yr	5 yr	10 yr	25 yr	100 yr
<i>e</i>	0.8315	0.8075	0.7881	0.7742	0.7808	0.7500
<i>b</i>	75.01	77.27	84.14	93.53	115.90	125.40
<i>d</i>	16.2	17.1	17.8	18.9	21.2	21.8

By fitting the various data points along a curve for an extended series of times of concentration, the curves can be used to create an intensity formula which represents the IDF curves. For Figure 9–4, the formula is:

$$i = \frac{b}{(Tc + d)^e} \quad (9-2)$$

where *i* = intensity (in./hr), and the *e*, *b*, and *d* coefficients are given in Table 9–2.

Numerous reference materials deal with the above application of the rational method. There are also several other methods for determining runoff rates for given drainage areas. Some of these other methods are listed in Table 9–3.

The rational method is generally considered applicable to drainage areas smaller than 200 acres. When dealing with contributing drainage areas larger than 200 acres, other methods listed in Table 9–3 may prove more applicable.

Design of Conveyance Conduits

Once a design storm is created in compliance with the maximum storm frequency, the peak flow of that storm can be used to design an adequate conduit. Two types of conveyance conduits are routinely used with drainage collection systems: closed conduits and open channels. Closed conduits include all types of sewer systems, including round pipes, square culverts, and other shapes of

Table 9–3. Common Runoff Determination Methods

Method	Description
Rational	Simplest and most commonly used method for urban storm drain systems. Usually limited to watersheds of less than 200 acres.
Area/runoff (regional regression equations)	Usually used to estimate peak discharges for larger watersheds in a specific geographical region at an ungauged site.
TR-20 and TR-55	NRCS hydrology methods for small to large ranging systems to estimate peak discharge and runoff hydrographs.
HEC-HMS	Used in large, complex drainage systems where storage, multiple tributaries, and hydraulic structures such as dams and weirs come into play, and where rainfall and runoff vary over both time and space.

conduits. Drainage-system open channels come in all sizes, from small drainage swales that are only barely noticeable to major open channels. Major open channels commonly encountered in floodplain management are discussed, from a design standpoint, later in this chapter. For drainage collection systems, the most common form of open channel encountered is the ordinary roadside ditch. Roadside ditches, like closed conduits, come in all sizes.

The actual design of urban storm sewers and open channels, or conduits, is a process of sizing the conveyance conduit to facilitate the estimated peak flow rate at that point in the system, considering the upstream drainage area(s). In using the rational method, as the drainage areas are summed moving downstream along the new or proposed storm drainage conduit, T_c is naturally increased. This reflects the dampening or attenuation of the storm flood wave as it propagates downstream through the conduits. To size the conduits themselves, it is common to use Manning's equation,

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2}, \quad (9-3)$$

where Q = flow (cfs), n = Manning's roughness coefficient = 0.011 to 0.013 for concrete and 0.022 to 0.035 for turf-lined ditches (see Table 7-1 for an expanded list of n -values), A = cross-sectional area (ft²), $R = A/P$ = the hydraulic radius (ft), and P = wetted perimeter (ft).

In parallel with the sizing of the conduits, the relative position of the resultant water surface profile should be examined. This is achieved by calculating the hydraulic grade line (HGL), starting at the most downstream point in the system, typically the storm drainage system outlet. Sometimes called a backwater calculation, the HGL represents the locus of elevations to which the stormwater would rise if open to atmospheric pressure, such as within piezometer tubes; it can be used to evaluate the adequacy of the design and identify areas of potential surface flooding. The difference in elevation of the water surface in successive conduit reaches reflects the friction and other minor losses in the conduits due to drops and bends, assuming gradually varied flow conditions. If the conduit has a slope equal to the friction slope, then the HGL will be parallel to the conduit. Considering Equation (9-3), the conduit slope, S , is equal to the friction loss, h_f , divided by the length of conduit, L , in full-flow conditions, ignoring minor losses. Solving for h_f , Equation (9-4) represents Manning's equation as utilized for determining the friction head loss of a conduit reach:

$$h_f = L \left(\frac{Qn}{1.49AR^{2/3}} \right)^2 \quad (9-4)$$

where h_f = head loss in conduit reach due to friction (ft), L = length of conduit reach (ft), and other terms are as previously stated.

It is also important for a designer to understand the relationship of the HGL to the energy grade line (EGL). The EGL represents the total

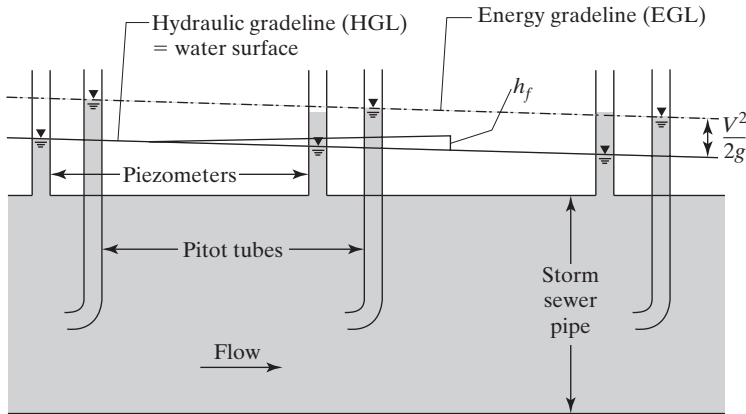


Figure 9-5
Relationship of the hydraulic grade line (HGL) and the energy grade line (EGL) in closed conduit systems under pressure.

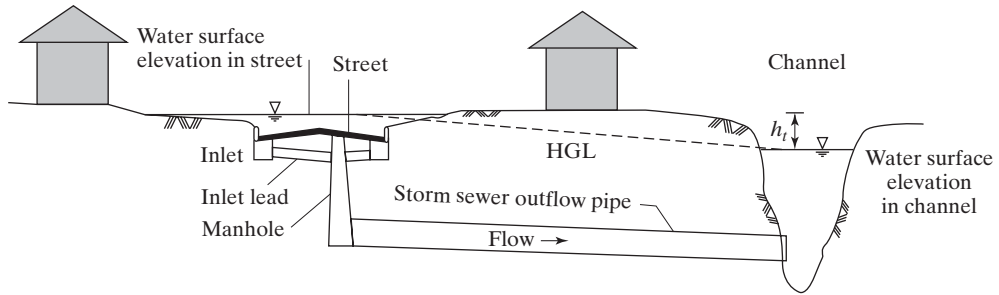
energy head, or the HGL plus the velocity head at any given point along a conduit computed by

$$EGL = HGL + \frac{V^2}{2g} \quad (9-5)$$

where V = velocity of flow (ft/sec), and g = gravitational constant (32.2 ft/sec²).

In a physical sense, the EGL would be represented by a series of pitot tubes (small tubes with their end openings pointed upstream) intercepting the kinetic energy of the flow, whereas the HGL is represented by piezometers that register only pressure head. Figure 9-5 illustrates the relationship between the HGL, the EGL, and a conduit (in this case a storm sewer pipe). Notice that if the starting water surface elevation of the downstream outlet of a conduit is below the soffit of the conduit itself, then the conduit may be flowing partially full. Conversely, if the outlet is fully submerged, then the conduit will be under pressure, at least at the outlet, assuming that the conduit is a pipe. Open channels behave similarly as closed conduits except that there are no bounding physical conditions that would cause an open channel to become pressurized, as typically occurs in a closed conduit such as a pipe. Consulting further references on the behavior of open and closed conduit systems is encouraged for full understanding of the behavior of such systems.

The relationship of the HGL to the surrounding environment is critical in the design of a storm drainage system. In a common urban environment, storm drainage systems usually exist as roadside ditches, roadside ditches in combination with an underground storm sewer, or a depressed curb-and-gutter street with an underground storm sewer. When two physical drainage facilities exist in conjunction with each other, such as with a depressed curb-and-gutter street and an underground storm sewer system, the storm sewer system acts as the primary drainage mechanism and the street as a secondary one (i.e., stormwater flows within the street as an open channel). This relationship demonstrates the importance of the relative position of the primary

**Figure 9-6**

Relationship of HGL to surface flooding.

drainage system HGL. For example, if a calculated HGL elevation is shown to be above the natural ground elevation along a project route, then surface flooding, even beyond the street, is indicated. Figure 9-6 is a simplified illustration that demonstrates the critical relationship between the HGL and the surface features of a project area, such as streets and structures (homes, buildings, etc.). If there is too much friction loss in a given sewer, surface flooding can occur whereby the streets are impassable to traffic and structures are in danger of flooding. What is ignored in the illustration is the other contributing losses due to the manhole, inlets, and inlet leads. These are very important factors in urban stormwater management design, and their design to provide connectivity between the primary and secondary drainage mechanisms is extremely important and is discussed later within this chapter.

EXAMPLE 9-1

STORM SEWER COLLECTION SYSTEM DESIGN

Figure 9-1 shows an actual drainage area map of a project area and urban storm sewer layout. Figure 9-4 shows the applicable IDF curves for this region, and Equation (9-2) provides a useful method for calculating the intensity values. The sizing of conduits for this storm sewer can be completed with a typical storm sewer design spreadsheet, as shown in Table E9-1(a). Complete the unfinished lines (shaded cells) of this spreadsheet (Manhole A6 to A3) using the data provided in the table and finish calculating the HGL elevations up to Manhole A9 in column 24 of the table. This will determine the correct size of each conduit needed to accommodate a given design storm and will calculate the HGL or water surface profile of the system. Given that this is a new storm sewer system in an existing urban area, many of the conduit characteristics are set in order to avoid conflicts with existing utilities. In the completion of this example, only the conduit size is to be determined as a design variable based on the other fixed characteristics (U/S and D/S elevations, lengths, and slopes). A minimum of 3 ft/sec shall be used in the storm sewer, assuming full-flow conditions in order to maintain a minimum self-cleaning velocity in the sewers. Use standard reinforced concrete pipe sizes in the design (0.5-ft diameter increments in size). Ignore minor losses of manholes.

Table E9-1(a). Storm Sewer Design Spreadsheet

Storm Sewer Design Spreadsheet																									
Project Name: Garden Villas Storm Sewer																			1.5 ft (18 in.) Span, Rise, or Dia						
Designer: JFT				HGL Starting WSEL: 31.00 ft										Minimum Conduit Size:					10 min						
Date: 12/12/2006				24-hr Storm Event: 2 yr										Minimum Tc to be Used:											
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	
U/S Manhole	D/S Manhole	Drainage Area		C	C × A	Sum C × A	Total DA (ac)	Tc (min)	I (in./hr)	Flow Q (cfs)	Conduit Characteristics										Actual Velocity (fps)	Friction Headlose (ft)	U/S Manhole HGL El (ft)		
		Elevation (ft)									Dimensions (ft)		Shape	Area (sq ft)	R (ft)	Length (ft)	Slope (ft/ft)	n	Capacity (cfs)						
		U/S	D/S								Span	Rise													
Line A																									
A9	A8	A9	7.44	0.45	3.35	3.35	7.44	19	3.88	13.00	34.00	32.60	2	2	Rnd	3.14	0.500	425	0.0033	0.013	13.01	4.14	1.40		
A8	A7	A8	4.77	0.45	2.15	5.49	12.21	21	3.71	20.38	32.10	31.15	2.5	2.5	Rnd	4.91	0.625	355	0.0027	0.013	21.26	4.15	0.87		
A7	A6	A7	11.36	0.45	5.11	10.61	23.57	23	3.55	37.66	29.65	29.00	4	4	Rnd	12.56	1.000	587	0.0011	0.013	47.90	3.00	0.40		
A6	A5	A6	8.48	0.45	3.82	14.42	32.05	26	3.34	48.16	28.71	28.00			Rnd			595	0.0012	0.013					
A5	A4	A5	10.42	0.45	4.69	19.11	42.47	29	3.15	60.28	27.24	26.75			Rnd			485	0.0010	0.013					
A4	A3	A4	3.48	0.45	1.57	20.68	45.95	31	3.04	62.91	26.75	26.40			Rnd			270	0.0013	0.013					
A3	A2	A3	1.09	0.45	0.49	21.17	47.04	32	2.99	63.29	26.10	25.39	4.5	4.5	Rnd	15.90	1.125	580	0.0012	0.013	68.95	3.98	0.60		32.42
A2	A1	A2	4.31	0.45	1.94	23.11	51.35	35	2.84	65.71	25.39	24.50	4.5	4.5	Rnd	15.90	1.125	530	0.0017	0.013	80.76	4.13	0.59		31.83
Line B																									
B3	B2	B3	2.5	0.65	1.63	1.63	2.5	11	4.81	7.82	27.60	26.60	1.5	1.5	Rnd	1.77	0.375	166	0.0060	0.013	8.17	4.43	0.92	33.24	
B2	B1	B2	0.87	0.45	0.39	2.02	3.37	12	4.67	9.42	26.60	25.75	2	2	Rnd	3.14	0.500	193	0.0044	0.013	15.05	3.00	0.33	32.32	
B1	A1	B1	1.35	0.70	0.95	2.96	4.72	13	4.54	13.43	25.28	24.25	2	2	Rnd	3.14	0.500	215	0.0048	0.013	15.69	4.28	0.75	31.99	
Line A																									
A1	Outlet	A1	5.42	0.45	2.44	28.51	61.49	38	2.71	77.32	22.18	21.92	5	5	Rnd	19.63	1.250	270	0.0010	0.013	81.00	3.94	0.24	31.24	

Notes: The Dimensions (span and rise) called for in the Conduit Characteristics will be the same in cases of round pipe. Other shapes can normally apply (i.e., box or oval sewers).

The Elevation data input in the Conduit Characteristics is for the flow lines of the conduits. U/S and D/S denote Upstream and Downstream, respectively.

The Actual Velocity is calculated based upon full-flow conditions for the design flow in Column 11.

SOLUTION

Once the fundamentals of the above equations are understood, the actual design development is rather straightforward. Referencing back to Figure 9–1 as a fundamental basis for the design, proceed from upstream to downstream in the calculations of the storm sewer sizing. Visualize the connectivity of the system. Line A goes from the Outlet to Manhole A9. Line B connects to Line A at Manhole A1 and continues up to Manhole B3. The following steps are worked for every conduit reach (a conduit reach being from one manhole junction to a connecting manhole junction) and apply to all lines in the spreadsheet.

1. Multiply the given C coefficient and contributing drainage area for the receiving upstream manhole location. Tabulate the result in column 6.
2. Sum the combined $C \times A$ values from the rational method in column 7, thus providing a weighted $C \times A$ value to account for different C values along the project route.

Note: For the last row (reach A1 to Outlet), add the upstream reaches in Line A and B together for the Total Drainage Area and Sum $C \times A$ (include its own drainage area, 5.42).

3. Determine the various intensity values from IDF curves or the related intensity formula using the determined T_c (in this example, the established T_c) and tabulate in column 10.

Note: The Design Storm is a 2-yr, 24-hr event, as stated at the top of the spreadsheet.

4. Calculate the flow rates, Q , for each design reach in column 11 using Equation (9–1).
5. At this point the conduit can be sized accordingly. A roughness of 0.013 is used, as the conduit shall be reinforced concrete pipe (see column 21). In this example, all conduits shall be round pipe and the conduit span and rise dimensions (columns 14 and 15) are the same. Since the geometry of the storm sewer system is fixed, as described above, the design entails finding a suitable pipe size that will meet flow and velocity criteria. Two means can be used at this stage to design the storm sewer size. The first method can entail the rearrangement of Equation (9–3) to explicitly find the pipe diameter in feet, since all other variables are known. Once this is done, the nearest larger standard pipe size can be used. The second method is simply by trial and error in the spreadsheet (try a size, see if it works, and adjust if needed). Both methods, using the explicit calculation of the conduit size or trial and error, work suitably well.

Remember, there are two basic checks to make when designing every conduit size:

- (a) Is the capacity of the sewer reach greater than or equal to the actual flow (column 22 compared to 11)?
- (b) Does the actual velocity of the sewer reach meet or exceed the minimum 3 ft/sec requirement?

The design intent is to use the smallest standard-sized reinforced concrete pipe that will meet these criteria, because this will provide the most cost benefit. In addition, if a conduit is too large for a given flow, the actual velocity of the flow in the sewer will be too low.

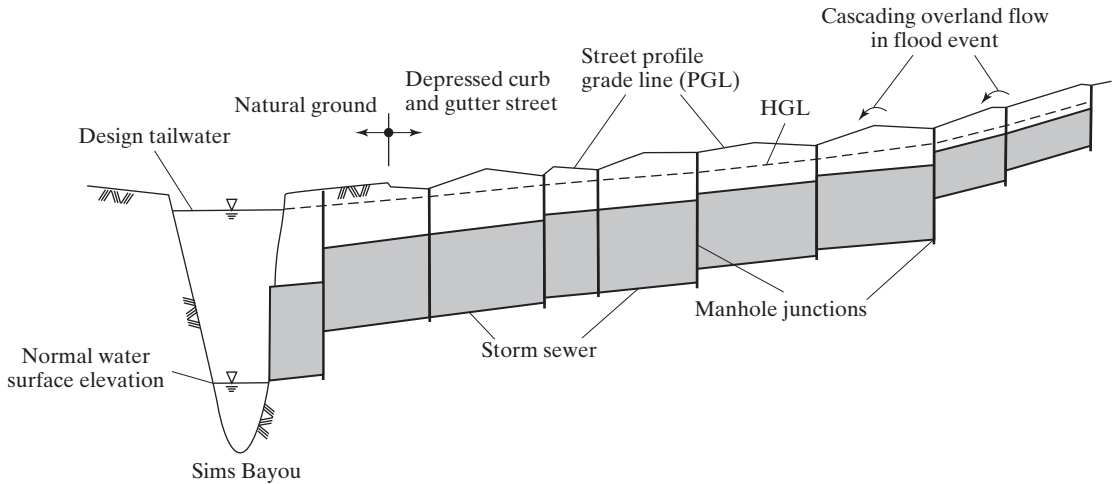
6. Once the pipeline diameter is determined, use the usual form of Manning's equation [Eq. (9-3)] to calculate flow capacity for the designed conduit (column 22). Dividing the actual flow (column 11) by the designed conduit area (column 17) gives the actual velocity in the conduit, assuming full-flow conditions.
7. After the conduits are sized, it is common then to calculate the HGL or water surface profile of the system. We do so by calculating the friction loss in each storm sewer reach and then adding those together, on a reach-by-reach basis, starting at the most downstream known water surface location—in this case, the outlet. Unlike the storm sewer system itself, which is designed from upstream to downstream, the HGL is determined from downstream to upstream. Minor losses, including manhole junction losses, can be employed for a more accurate HGL computation, but in many instances, and in this example, the minor losses are ignored.

In closed conduit systems, the friction loss in each reach can be found with Equation (9-4). The HGL is formed by adding those losses, on a reach-by-reach basis, to the starting water surface elevation at the outlet—hence the importance of defining a proper starting water surface elevation at the outlet (31.00 ft in this example—see the top of the spreadsheet). Given the starting tailwater elevation relative to the storm sewers, the sewers are flowing full in this example, and the friction loss is computed in a straightforward manner, as described above.

In completing the design as stipulated above (using the smallest standard-sized reinforced concrete pipe, in half-foot increments, that has sufficient capacity and yields at least the minimum full-flow velocity required), the design solution for the reaches' left bank is tabulated in Table E9-1(b). A profile plot of the storm sewer design for Line A is included in Figure 9-7, and important considerations of overland flow are discussed following this example.

Table E9-1(b). Design Solutions for Storm Sewer Reaches

Reach			
U/S Manhole	D/S Manhole	Span and Rise (ft)	U/S Manhole HGL El (ft)
A9	A8	2	36.49
A8	A7	2.5	35.09
A7	A6	4	34.22
A6	A5	4	33.82
A5	A4	4.5	33.15
A4	A3	4.5	32.70

**Figure 9-7**

Typical profile plot of urban storm sewer system.

The overall way a storm sewer design works hand-in-hand with surface drainage features is critical to the performance of the combined subsurface and surface drainage components (i.e., the underground conduits and the above-ground drainage mechanisms like the surface streets). Figure 9-7 is a profile plot of the design results of Line A in Example 9-1. Notice that the HGL is suitably below the natural ground and street gutter lines, thus indicating that surface flooding is not anticipated for that particular design event, assuming the inlets are designed adequately, as discussed later in this chapter. Also notice that the surface streets are graded to allow for an overland flow path of stormwater, in an extreme storm event that would exceed the designed capacity of the storm sewer system, to flow to the receiving outfall, in this case a bayou, in a cascading fashion over the street profile grade line (PGL) high points. Referring both to Figures 9-6 and 9-7 gives a visual representation of the relationships between the storm sewer design, the position of the HGL of the sewer system, and the design of the overland flow mechanisms such as the surface streets.

In designing storm sewer systems, it is important to consider the effect of the downstream starting water surface elevation at the outlet or outfall. The water surface elevation chosen for the outlet dictates the regional storm's effect on HGL height in the local system. This elevation must have some relation and may be used as provided by other studies and analyses or as determined from approximations, if acceptable, using normal depth calculations within the outlet receiving channel. In many instances, the design criterion allows for simplified assumptions pertaining to starting water surface elevations at the outlet for the purposes of computing the HGL along a designed storm sewer system. This may be the soffit (top or crown) of the last reach of pipe at the outlet or even perhaps some arbitrary elevation as prescribed by a governing flood control agency. In any case, the HGL is computed upstream

throughout the storm sewer system to detect instances where significant head loss may exist. Increasing the size of the conduit increases the hydraulic radius of the conduit, thereby reducing the friction loss and the slope of the HGL. In cases where certain HGL criteria must be met, a trial-and-error type of approach is often exercised to finalize a design that satisfies the design intent.

Minor Losses in Storm Sewers

Manhole and junction losses as well as bend losses may be considered in the development of the HGL profile. In some cases, these losses are considered minor and insignificant; in other cases, especially where relative velocities are rather high, the losses may be considered significant and therefore added to the conduit friction losses at the junctions. Equation (9-6) represents the equation for the calculation of manhole/junction losses. Notice the primary influencing parameter in these equations is the velocity term. When the velocity is rather low, these losses are indeed very minor (thousandths of a foot); however, they do add up and can exacerbate a potential flooding problem due to high-HGL problems, such as illustrated in Figure 9-6. In general, it is desirable to keep the minimum velocities within conduits above 3 feet per second. Maximum velocities should not exceed 8 feet per second without due consideration of the use of energy-dissipation features to control the effects of high velocities such as erosion. Again, local design guidelines generally dictate this controlling criterion.

In some design circumstances, the friction coefficient, n , is used to consider the minor losses anticipated. In Example 9-1, had the friction coefficient been 0.011, a reasonable n -value found in laboratory case studies, in lieu of the used 0.013 value, the total friction loss would have been reduced over 1.5 ft throughout the entire Line A system. Hence the n -value can be used to aid the overall design in terms of trying to more accurately estimate the total loss through the system when minor losses (bends, manhole junctions, and inlet losses) are ignored.

When manhole junction losses are desired to be considered, we may use the equation

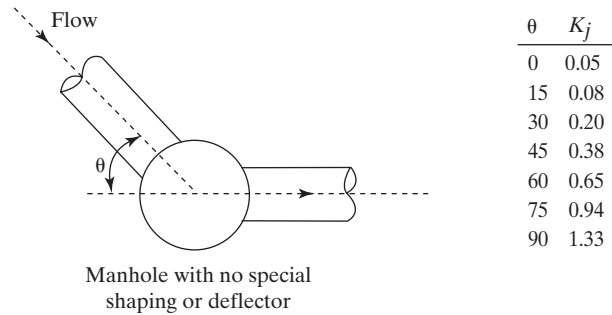
$$h_j = K_j \left(\frac{V^2}{2g} \right) \quad (9-6)$$

where h_j is the manhole/junction loss without a curved deflector (ft), V is the velocity through the manhole in ft/sec, g = gravitational constant (32.2 ft/sec²), and K_j is as shown in Figure 9-8.

Inlets Used with Storm Sewers

Equally important as the design of the conduits is the design of inlets and other stormwater capture facilities. An inlet is simply a stormwater capture feature that physically receives flow from overland sheet flow or more channelized flow such as within a gutter, and the importance of properly sized inlets cannot be overstated. After all, the conduit system may be sized adequately for a certain design event, but if the stormwater cannot enter the conduits at the

Figure 9–8
Manhole junction
loss coefficients.



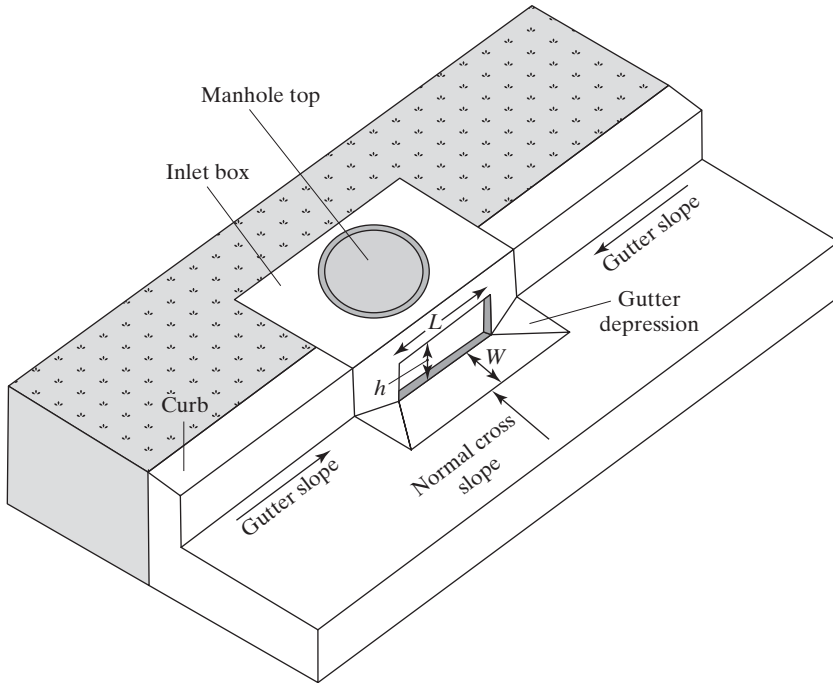
rate designed for, then the hydraulic connectivity of the system is compromised. This is especially true of depressed curb-and-gutter street sections using conventional on-grade and sag inlets. Inlets work hand-in-hand with the surface streets in urbanized storm sewer systems. As previously mentioned, the storm sewers act as the primary drainage system while the surface streets act as the secondary system. Those stormwater flows not facilitated by the storm sewers are conveyed overland by the street system. Inlets provide the vital function of hydraulic connectivity between the primary and secondary systems. Much study has been done on the subject of inlet behavior, and studies are still ongoing. The inlet capacity is designed based upon allowable street ponding widths, street gutter flow rates, street physical features such as slope and friction value, and the inlet physical features. It is highly recommended that inlet computations be precisely considered using established guidelines such as those developed by the Hydraulic Engineering Circular No. 22 (HEC 22) as published by the Federal Highway Administration (FHWA).

There are all types and shapes of inlets. In general, inlets act as weirs until submerged, and then they transition to act as orifices. In curb opening inlets at a roadway depression, as illustrated in Figure 9–9, the weir and orifice behavior transitions at depths between 1.0 and 1.4 times the inlet opening height. In these cases, the inlet behavior should be analyzed using both the weir and orifice equations, and the lesser of these two capacities computed should be used in the design (a conservative approach). Test studies have shown that inlet physical features such as inlet throat depressions and grate types greatly affect the actual performance of an inlet. For these reasons, it is recommended to refer to detailed guidelines as previously mentioned or to check with local governing agencies, which likely will have published allowable inlet capacities for those types of inlets commonly utilized in the project area. Absent of these detailed procedures, inlet behavior may be approximated using weir and orifice equations, as depicted in Eqs. (9–7) and (9–8); yet, due consideration must be given to the other parameters that may limit inlet capacities, such as inlets on steep grades, which induce the likelihood of substantial inlet bypass flow:

$$Q = C_w(L + 1.8W)d^{1.5} \tag{9-7}$$

where Q = weir condition inlet flow (cfs), C_w = weir coefficient = 2.3 (if no gutter depression, $C_w = 3.0$), L = length of curb inlet opening (ft), d = depth

Figure 9-9
Standard curb
inlet with gutter
depression at a
sag in a roadway.



of water at the inlet curb measured from the normal cross slope of the pavement (i.e., depth at curb just upstream of the inlet out of the gutter depression) (ft), and W = lateral width of inlet throat depression (ft).

$$Q = C_o h L (2gd_o)^{0.5} \quad (9-8)$$

where Q = orifice condition inlet flow (cfs), C_o = orifice coefficient = 0.67, L = length of curb opening (ft), d_o = effective head on the center of the inlet throat (i.e., the depth of water measured to the center of the inlet throat considering the gutter depression depth) (ft), g = acceleration due to gravity (32.2 ft/sec²), and h = height of inlet throat curb opening (ft).

The importance of the behavior of the primary and secondary drainage systems, working collectively, cannot be overemphasized. When surface flooding does occur, ponded sections within roadways and other areas act as storage junctions. These junctions detain stormwater flows until the primary drainage system can facilitate that stored volume. In cases where there is insufficient storage volume on the surface for a particular storm event, then the water surface elevation (WSEL) of the flooded secondary system, commonly a street, increases to a point in which structural flooding of adjacent homes and buildings may occur. Furthermore, stormwater typically flows overland between the surface storage junctions. This is a very common occurrence in urban drainage systems where overland flow is conveyed between surface storage junctions, which may exist as street intersections, for example. In these cases, the relative position of the overland flow routes in a project area

must be fully understood, so as to limit the potential for build up of flooded WSEL to the point at which the structural flooding of homes and businesses occurs. Such surface conduits (i.e., the streets) may be considered using weir equations or even calculated as conventional open channels. Mother Nature developed flow patterns along our surface topography long before we induced urbanized development upon our lands. These natural flow patterns must be recognized and respected, as there is always another natural storm that may or eventually will occur above and beyond that which was planned for in the design development of our urban drainage systems.

9.3 Fundamentals of Culverts

DESIGN OF CULVERTS

Culverts are commonly referred to as those conveyance conduits constructed within embankments for the purposes of supporting a roadway, a railroad, or for the particular functionality of the embankment itself, such as would be the case in a levee. Culverts are different than storm sewers in that they are typically much shorter (although there are cases of long-span culverts) and are exposed at both ends to accommodate unobstructed inflow and outflow of storm water. The simple culvert crossing of a street connecting two road-side ditches is one of the most common culvert applications readily identified. Culvert crossings, as they are commonly known, come in all shapes and sizes. A variety of materials are often used in culvert construction, in a variety of shapes, such as circular, box, and elliptical. As needed to convey a given amount of stormwater flow, given particular depth requirements, multiple barrels of culverts may be used at a single location. While appearing simple, culvert behavior and culvert design are deceptively complex.

Referring to Figure 9–10, the factors that affect culvert design include the headwater (HW), the tailwater (TW), the length (L), the size or diameter (D), the slope (S), and the friction factor (n). Many other factors also affect the performance of culverts, such as the inlet and outlet geometry, which would include the use of headwalls and even the shape of the culvert edges. The basic driving culvert design parameter is considered to be the allowable headwater depth. After all, the intent of a culvert is to allow stormwater to pass through an embankment at a planned rate, which relates to an allowable headwater depth on the upstream side of the culvert, given an established

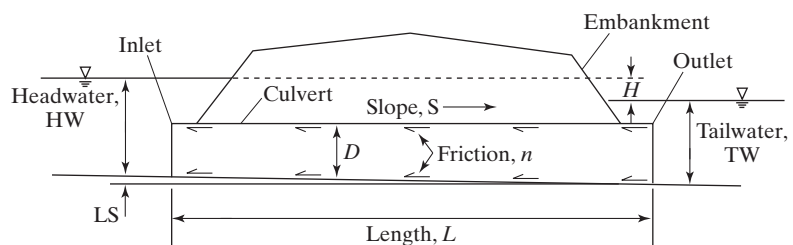


Figure 9–10
Relationship
of culvert
features and
characteristics.

tailwater condition downstream of the culvert. Looking at Figure 9–10, the simple balanced equation for comparing the water surface elevations on both sides of the culvert is

$$HW + LS = TW + H \quad (9-9)$$

where HW is the upstream headwater at the inlet of the culvert, LS is the product of the length multiplied by the slope of the culvert, TW is the downstream tailwater at the outlet of the culvert, and H is the total head loss through the culvert, as described in further detail below.

Culverts are generally divided into two classifications: **inlet controlled** or **outlet controlled**. Inlet-controlled culverts are governed by the inlet of the culvert, and the discharge through the culvert is predominately affected by the culvert entrance. In a simplistic sense, inlet control exists when the discharge through the culvert itself can be at a greater rate than what is capable of entering into the culvert at the inlet. Inlet-controlled culverts are not affected so much by the internal culvert characteristics, such as roughness and length, as by the headwater depth, entrance geometry and configuration, and type of inlet edge. Culverts do not flow full throughout their entire length under inlet control conditions, and generally flows are supercritical in the culverts. If the culvert application is in an environment of relatively steep slope, inlet control is more likely. Figure 9–11 illustrates an example of inlet-control conditions for a culvert; other inlet-control flow conditions exist as well (see reference material for a more detailed description of other flow conditions).

Inlet control can be somewhat involved in terms of the calculations employed due to the partially full-flow conditions of the culvert. Many publications document the behavior of culvert conditions in a variety of applications. As would be typical, inlet-control conditions require the use of tables, charts, nomographs, or other available reference materials from the Federal Highway Administration (FHWA) or other sources that will aid the designer in defining the behavior of the culvert in terms of hydraulic performance based upon extensive research. The design of inlet-controlled culverts can be complex for the reasons stated above, and in lieu of using the described published culvert behavior tables, charts, and so on, it is common to apply readily available software programs, such as HEC-RAS, as described later in this chapter.

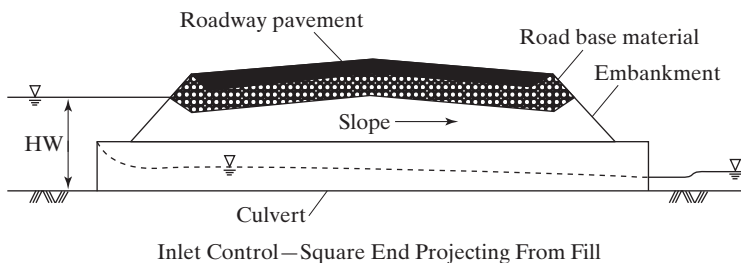
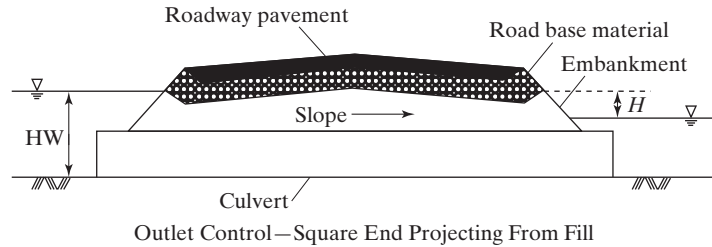


Figure 9–11
Example of inlet control conditions for a culvert crossing.

Figure 9-12

Example of outlet control condition for a culvert crossing.



Outlet control for culverts exists if the rate of flow that can enter the culvert is greater than that possible through the culvert. In outlet-controlled culverts, the physical characteristics of the culvert play a significant role, as do the tailwater conditions for a particular storm event being applied. If the culvert application is in an environment of relatively mild slope, outlet control is more likely to govern. Generally, culverts flow full throughout all or most of their length in outlet-control conditions, and the flow is either subcritical or under pressure throughout the culverts. Figure 9-12 illustrates an example of outlet-control conditions for a culvert; however, outlet-control circumstances can exist whereby the culvert outlet is not submerged (see reference material for a more detailed description of other flow conditions).

Looking at Figure 9-12, the tailwater is above the outlet crown of the culvert pipe (the crown of a pipe is the inner top of the pipe, often called the pipe soffit). If the tailwater is below the crown of the pipe, then outlet-control conditions can still exist, but the calculation of the tailwater depth will be more involved. For the condition shown in Figure 9-12,

$$H = H_{\text{ent}} + H_{\text{exit}} + H_f \quad (9-10)$$

where H_{ent} equals the entrance head loss in feet, H_{exit} equals the exit head loss in feet, and H_f equals the friction head loss in feet.

The entrance loss is computed by,

$$H_{\text{ent}} = K_e V^2 / 2g \quad (9-11)$$

where K_e is an entrance loss coefficient and the other terms are as applied previously. Table 9-4 provides several commonly applied entrance loss coefficients.

The exit head loss computation can be simplified by assuming the downstream culvert end empties into a nonflowing body of water, in which case the exit head loss is computed by taking the full velocity head at the exit of the culvert ($V^2/2g$, using V as the velocity in the culvert). A more accurate means is to recognize the downstream velocity in the channel, downstream of the culvert. Therefore, the exit head loss is computed by

$$H_{\text{exit}} = (V^2/2g)_{\text{culvert}} - (V^2/2g)_{\text{channel}}, \quad (9-12)$$

where V equals the velocity of flow in the culvert or downstream channel as appropriate in feet per second, and g equals the gravitational constant of 32.2 ft/sec². H_f is the friction loss within the culvert and is calculated by Equation (9-4).

Table 9-4. Culvert Entrance Loss Coefficients, K_e

Type of Structure and Design of Entrance	Coefficient, K_e
Concrete Pipe	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall with wingwalls	
Socket end (groove end)	0.2
Square end	0.5
Rounded end	0.2
Mitered to conform to fill slope	0.7
Corrugated Metal Pipe or Pipe Arch	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls, square edge	0.5
Mitered to conform to fill slope	0.7
Concrete Box	
Headwall parallel to embankment (no wingwalls)	
Square edged on 3 edges	0.5
Rounded on 3 edges	0.2
Wingwalls at 30 to 75 degrees to box	
Square edged at crown	0.4
Crown edge rounded	0.2
Wingwalls at 10 to 30 degrees to box	
Square edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square edged at crown	0.7

Equation (9-9) in conjunction with Equation (9-10) provides a simple approach to solving outlet-control culvert conditions when the tailwater has been computed by means of downstream normal depth computations or has been provided by other means. The sizing of the culvert or culverts is based upon a simple iterative process whereby various culvert sizes for a given length are used, losses are computed, and the resultant headwater is determined. If unsatisfactory in terms of headwater height, enlarging the culvert or culverts or adding more culvert barrels may be employed.

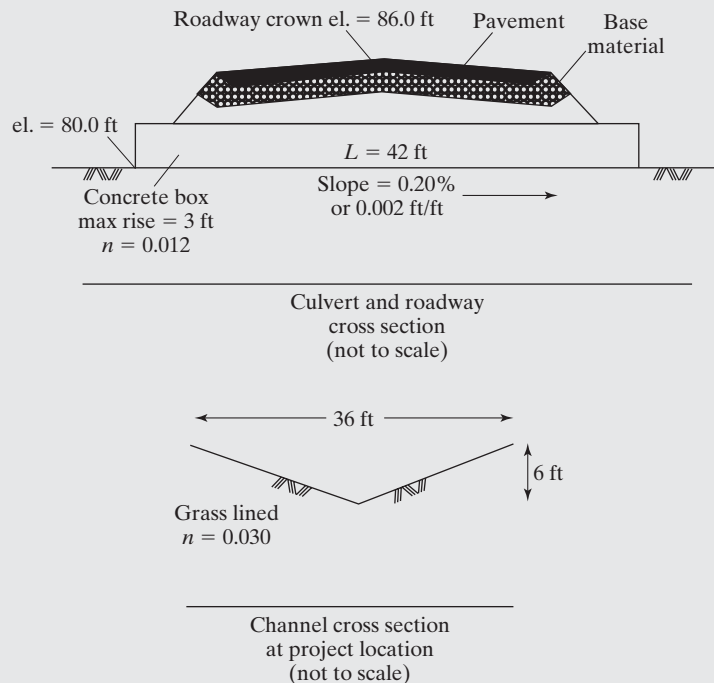
In looking at the design of a culvert system at a given location suspected of being under outlet-control conditions (i.e., a very mild or flat sloped environment), a basic starting point is determining the actual tailwater elevation at the given location. This can be achieved by many means. If a known nearby studied stream is associated with the local drainage network, perhaps a backwater analysis using HEC-RAS would be warranted. But in many cases where few or no data exist to readily establish a given tailwater elevation at a given location, it is common to compute the runoff at the location of interest using the rational method or some unit hydrograph method as described in Chapter 6 and then use a normal depth calculation using Manning's equation [Eq. (9-3)] to determine the depth of flow for that particular location.

EXAMPLE 9-2**DESIGN OF CULVERT CROSSING**

A new culvert is to be designed and constructed replacing an existing culvert in a small channel at a location which receives drainage from 160 acres of park land near Houston, Texas. The design criteria require that the culvert be sized such that a 10-yr synthetic design storm for the project area will not cause stormwater to overtop (i.e., go over) the roadway, as shown in Figure E9-2(a). Due to the two-lane roadway pavement and pavement base material, the maximum rise or height of a culvert to be used is 3 ft. Note that a box culvert must be utilized, since a 3-ft-diameter round pipe culvert exists in this location now and has proven inadequate.

- Use the rational method with $T_c = 18$ min., $C = 0.18$ (for park land), and the IDF curves shown in Figure 9-4 along with Equation (9-2) for a 10-yr storm.
- Establish the tailwater (TW) depth in the channel by calculating the normal depth at the downstream end of the culvert using the Manning's equation applied to the channel cross section shown in Figure E9-2(a).
- Establish the allowable total head loss, H .
- Size the culvert using a 3-ft maximum rise or height. Use a square-cut end entrance projecting from the fill of the embankment.

Figure E9-2(a)
Culvert crossing
characteristics.



1. Compute the runoff to the culvert crossing location.

SOLUTION STEPS

$$T_c = 16 \text{ min,}$$

$$Q = (CA) \cdot i,$$

$$I = 6.0 \text{ in./hr [from IDF curve, Fig 9-4; or Eq. (9-2)],}$$

$$A = 160 \text{ acres,}$$

$$C = 0.18 \text{ (see Chapter 6),}$$

$$Q_{\text{peak}} = (0.18)(160)(6.0),$$

$$= 172.8 \text{ cfs.}$$

2. Calculate the normal depth in channel to determine TW .

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2},$$

where

$$Q = Q_{\text{peak}} = 172.8 \text{ cfs,}$$

$$n = 0.030 \text{ (friction coefficient in the channel),}$$

$$S = 0.002.$$

Solve for $AR^{2/3}$.

$$AR^{2/3} = \frac{172.8}{\left(\frac{1.49}{0.030}\right)(0.002)^{1/2}},$$

where

$$AR^{2/3} = 77.797,$$

$$A = \text{area of flow,}$$

$$P = \text{wetted perimeter,}$$

$$R = A/P.$$

Considering the triangular roadside channel with 3:1 side slopes as shown in Figure E9-2(b), and using the calculated $AR^{2/3}$ above, calculate the uniform depth of flow, d , through iteration.

$$d = 4.09 \text{ ft.}$$

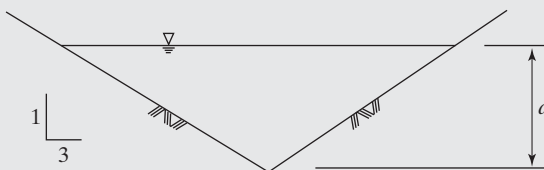


Figure E9-2(b)
Channel depth
geometry.

3. Establish the maximum allowable total head loss, H , using the known maximum upstream water surface elevation allowed.

The culvert is an outlet-control scenario because, given the limitation of a maximum 3-ft rise or height of our box culvert, the outlet is submerged ($3 < 4.09$), and we have an extremely flat slope along the channel (0.002 ft/ft), which will result in a higher HW relative to the conduit, due to the total head loss through the system; therefore, the conduit barrel is under pressure its entire length. Additionally, given the slope and flow for the channel, the critical depth can be calculated as 2.91 ft (see Chapter 7 for description of critical depth), which is below the actual depth of flow calculated above, which indicates the flow is subcritical in the channel.

The upstream invert or flow-line elevation of the culvert is 80.0 ft. Therefore, the downstream culvert flowline is

$$80 - 42(0.002) = 79.92 \text{ ft},$$

where 0.002 is the slope of the culvert and 42 is the length of the culvert in feet. The TW elevation (the downstream water surface elevation) is then simply the elevation of the downstream culvert flow line plus the normal depth, d , of flow in the channel.

$$TW = 79.92 + 4.09 = 84.01 \text{ ft}.$$

The maximum HW elevation (the upstream water surface elevation) is 86.00 ft, since a project criterion is to not have roadway overtopping. Therefore,

$$\text{allowable } H = 86.00 - 84.01 = 1.99 \text{ ft}.$$

4. Size the culvert.

Using a 3-ft maximum rise in box size, try different box spans (widths) of standard size (box culverts are usually width $\times$ rise—i.e., a 6×3 would be a 6-ft span or width by a 3-ft rise or height). Standard box spans are in 1-ft increments. Try a 6 ft $\times$ 3 ft box.

$$Q = VA \text{ (continuity equation ignoring storage),}$$

$$172.8 = V(6 \times 3),$$

$$V = 9.60 \text{ ft/sec through the culvert,}$$

and for the channel with area $A = 50.18 \text{ ft}^2$ calculated using the flow depth of 4.09 ft,

$$V = 172.8/50.18,$$

$$V = 3.44 \text{ ft/sec in the channel downstream of the culvert.}$$

Calculate the entrance head loss using the culvert velocity.

$$\begin{aligned}
 K_e &= 0.5 \text{ (Table 9-4, square-cut end projecting from fill),} \\
 H_{\text{ent}} &= K_e \left(\frac{V^2}{2g} \right) \\
 &= 0.5(1.43) \\
 &= 0.72 \text{ ft.}
 \end{aligned}$$

Calculate the exit head loss using both the culvert velocity and the downstream channel velocity.

$$\begin{aligned}
 H_{\text{exit}} &= (V^2/2g)_{\text{culvert}} - (V^2/2g)_{\text{channel}} \\
 &= [9.60^2/2(32.2 \text{ ft/sec}^2)] - [3.44^2/2(32.2 \text{ ft/sec}^2)] \\
 &= 1.25 \text{ ft.}
 \end{aligned}$$

Calculate the friction loss through the culvert.

$$\begin{aligned}
 H_f &= L \left(\frac{Qn}{1.49AR^{2/3}} \right)^2 \\
 &= 42 \left(\frac{172.8 \times 0.012}{1.49(6 \times 3) \left(\frac{6 \times 3}{2(6) + 2(3)} \right)^{2/3}} \right) \\
 &\quad \left(R = \frac{A}{P}, \text{ culvert is flowing full} \right) \\
 &= 0.25 \text{ ft,} \\
 H &= H_{\text{ent}} + H_{\text{exit}} + H_f \\
 &= 0.72 + 1.25 + 0.25 \\
 &= 2.22 \text{ ft} > 1.99 \text{ ft allowable.}
 \end{aligned}$$

Increase span of culvert to reduce the total head loss, H .

Try 7×3 culvert and use the same calculations as before to solve:

$$\begin{aligned}
 H_{\text{ent}} &= 0.53 \text{ ft,} \\
 H_{\text{exit}} &= 0.87 \text{ ft,} \\
 H_f &= 0.17 \text{ ft,} \\
 H &= 0.53 + 0.87 + 0.17, \\
 &= 1.57 \text{ ft} < 2 \text{ ft allowable, OK.}
 \end{aligned}$$

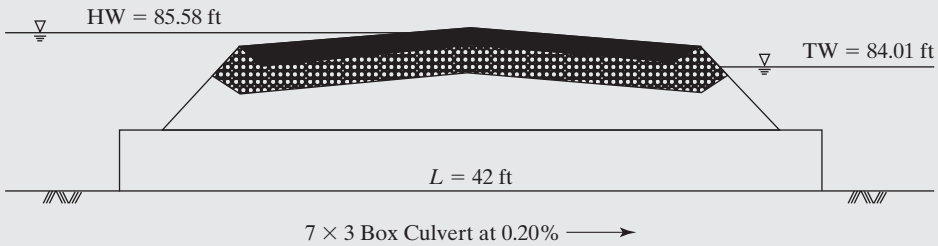


Figure E9-2(c)

Final design of culvert crossing.

5. Check the final design.

$$\begin{aligned}
 HW \text{ elevation} &= TW \text{ elevation} + H \\
 &= 84.01 + 1.57 = 85.58 \text{ ft} < 86.0 \text{ ft, OK.}
 \end{aligned}$$

A note of caution on the use of *elevations* versus *depths*. In the above calculation, we are actually using the *HW* and *TW elevations*. In Equation (9-9), the terms used to balance the equation are *depths*. The final design is illustrated in Figure E9-2(c).

Due to the relatively high velocity through the box culvert (8.23 ft/sec), erosion-control features should include concrete slope paving or broken rock of sufficient size, gradation, and weight to not be rolled downstream by the velocity of the water, placed strategically along the channel slope face.

This same culvert configuration can be modeled in HEC-RAS with very much the same results. As can be seen from the culvert output window in Figure 9-13, the entrance, exit, and friction losses are exactly the same as calculated above. The minor difference in upstream water surface elevation is due to the modeling methodology used in HEC-RAS for culvert routines. RAS calculates the water surface elevation at the first cross section upstream of the culvert by computing the water surface that corresponds to the energy for the given flow rate. The input is rather straightforward, and, following the standard culvert modeling procedures outlined in the HEC-RAS User's Manual available online, the same input data as described in the above example will yield the same results illustrated in Figs. 9-13 and 9-14. (*Note:* The exact cross-section locations in HEC-RAS will vary the water surface elevation results slightly.)

Complex Culvert Design and Analysis Using HEC-RAS

As previously stated, culvert design can be deceptively complex. For simple applications, as illustrated above in Example 9-2, hand calculations will work very well and suitably for a given scenario. There are numerous good reference manuals for culvert design with nomographs for various culvert

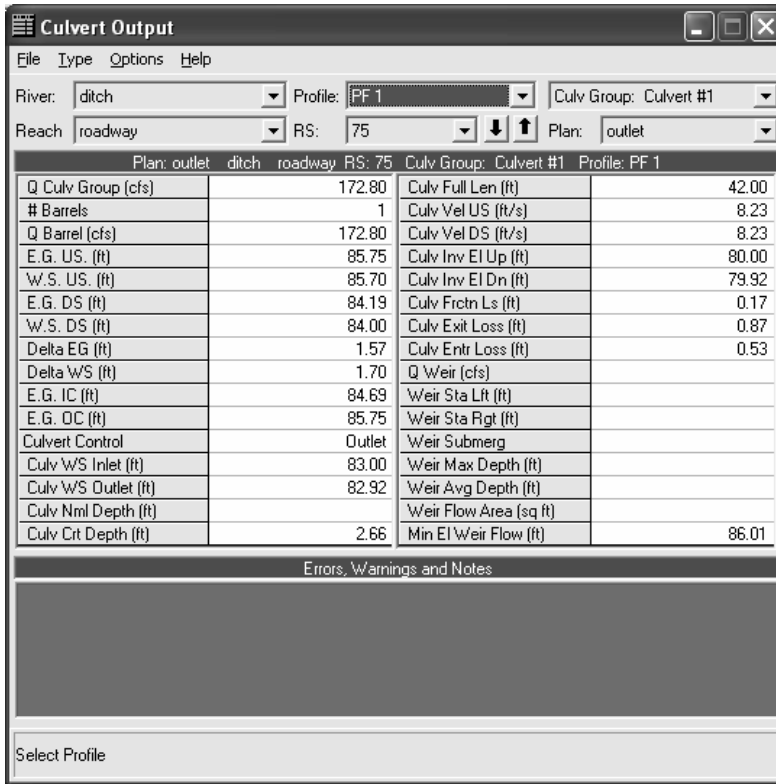


Figure 9-13
HEC-RAS culvert
output table of
Example 9-2.

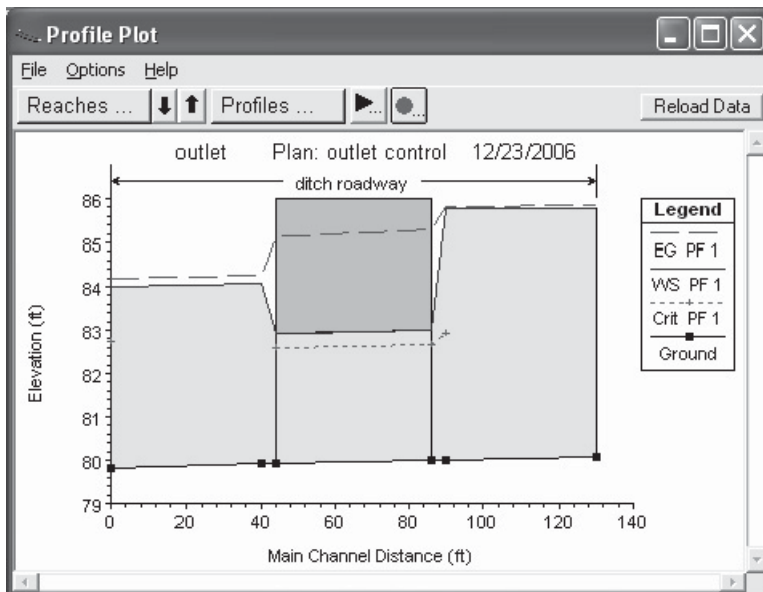


Figure 9-14
HEC-RAS
profile plot of
Example 9-2.

configurations and characteristics. The FHWA is an excellent resource for such information, as are the various State Departments of Transportation throughout the United States. The procedures within these published reference manuals are intended to aid a designer and avoid the use of more arduous computational procedures by hand. Many culvert flow scenarios can exist—partially full with inlet control, partially full for some culvert length with outlet control, and so on. For this reason, the discussion and examples herein are intended only to serve as an introduction to the meticulous issues facing a designer when facing a culvert problem, and it is strongly recommended that the cited reference material be examined for a much expanded discussion on the theory behind culvert behavior and design.

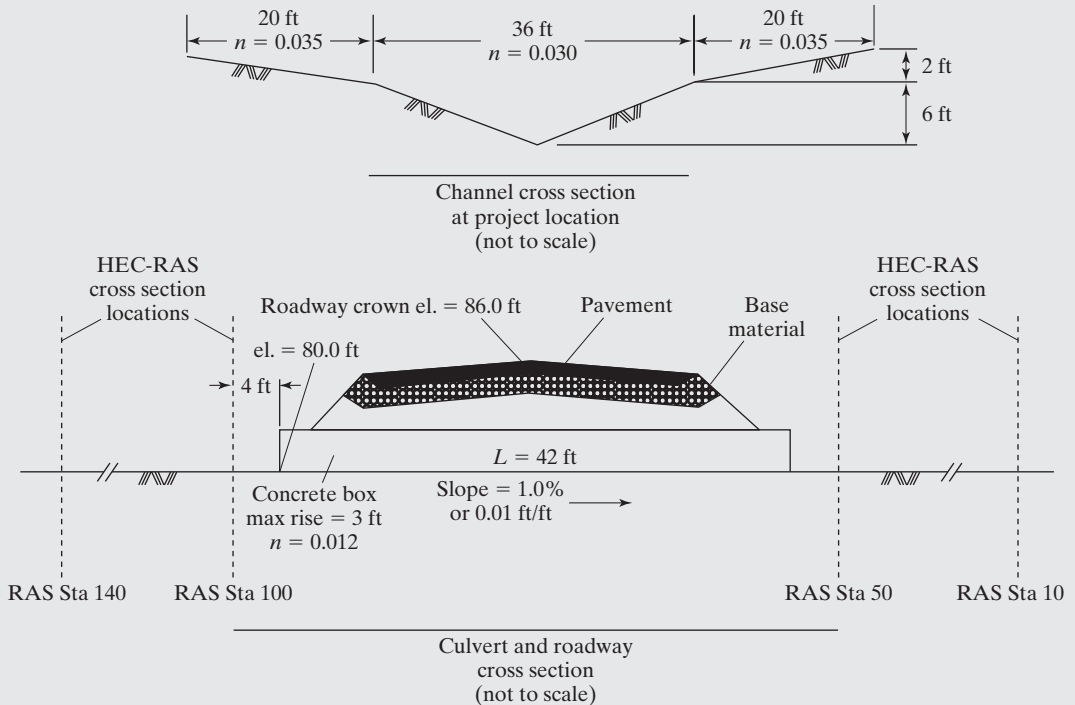
Considering the complex behavior of culverts and culvert systems, an excellent tool that is readily available, relatively simple to apply, and is commonly used is the HEC-RAS computer program discussed in Chapter 7. HEC-RAS is free and, once the user is familiar with the culvert routines included within the software, it is relatively easy to use. The main benefit of using HEC-RAS, or some other sort of culvert analysis software, is that the various flow regimes that exist at any given design location can be readily analyzed, and the complex calculations that are required for various flow scenarios are easily accommodated by the program. In addition, the output of the software provides a great deal of beneficial information that is useful in refining a given design.

Not all culvert applications are simple ones, 90 degrees to the embankment. Culvert skews are common, as are various other design elements, such as complex stream flows, that necessitate a more complex analysis. The HEC-RAS User's Manual is a good reference source for some of these applications. In keeping with the basic design and analysis approach presented thus far, Example 9-3 involves the analysis of the same culvert crossing as described in Example 9-2 but with a steeper slope (1%) of the man-made drainage channel.

EXAMPLE 9-3

ANALYSIS OF CULVERT CROSSING USING HEC-RAS

Using the same basic cross-sectional channel geometry and roadway/culvert configuration as presented in Example 9-2, but now with a channel slope of 1% (in lieu of 0.02% in Example 9-2), analyze the behavior of the 7×3 box culvert crossing for the same 10-yr storm event ($Q = 172.8$ cfs). Investigate the ability to alter the design of the culvert crossing to a 6×3 box for the benefit of cost savings; however, overtopping of the roadway is not allowed (i.e., flow over the roadway is not allowed). Refer to Figure E9-3(a) for HEC-RAS cross-section locations. These locations are simplified for this example. In true applications, due consideration should be given to contraction, expansion, ineffective flow, and so on in the development of cross-section locations. All channel cross sections are parallel, as there are no curves in the channel (the channel, right overbank, and left overbank have the same distances to the downstream section at all locations). The culvert crossing has no skew. Due to the slope of the channel

**Figure E9-3(a)**

Culvert crossing information for use in HEC-RAS model.

and culvert system as well as the previously known behavior of the 7×3 box culvert, it is suspected that the culvert is under inlet control.

1. Create a project in HEC-RAS titled *culvert example* and with a file name of *culvert.prj* (File → New Project → type *culvert.prj*)
2. Create a new geometric data file with 4 cross sections and the culvert. In the geometry file, create a River called *Ditch* and a Reach called *Roadway*. Enter 4 cross sections (2 upstream and 2 downstream of the culvert), with river stations 10, 50, 100, and 140.

For these cross sections, calculate the flow-line and top bank elevations based on maintaining the upstream invert section 4 feet upstream and downstream of the culvert (stations 100 and 50, respectively). Refer to Figs. E9-3(a) and E9-3(b) for cross-section placement, and base all elevations on the upstream culvert invert elevation of 80.00 ft. The upstream end of the culvert is technically at river station 96 ($100 - 4$). Remember that in RAS, stationing is upstream to down, and cross sections are viewed looking downstream. The channel slope and all resultant cross-section elevation changes are based upon a 1% slope or gradient. The centerline station for the cross sections and culvert is set at station 18. Other data are as presented in Figure E9-3(c).

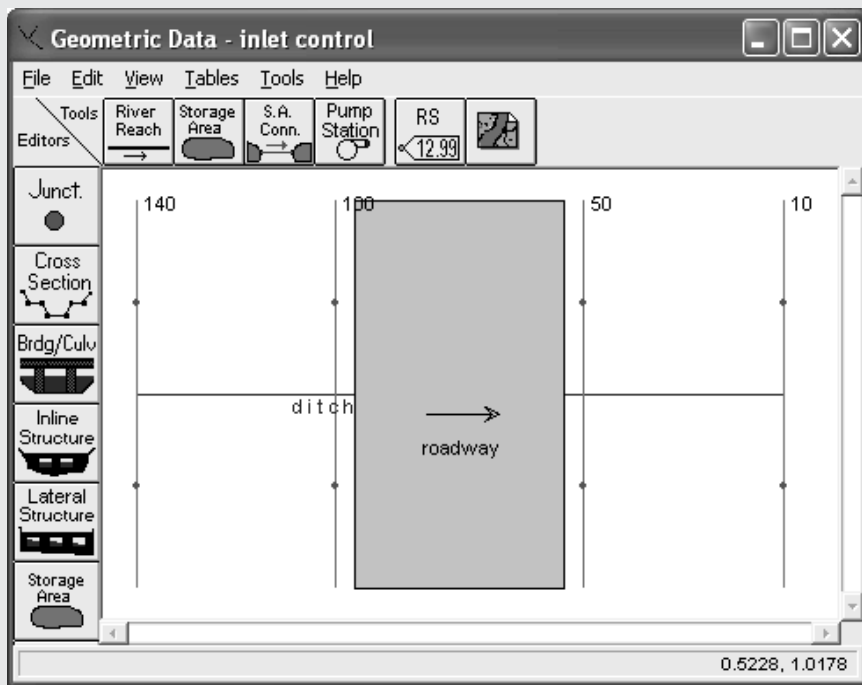


Figure E9-3(b)
Geometric data plan view.

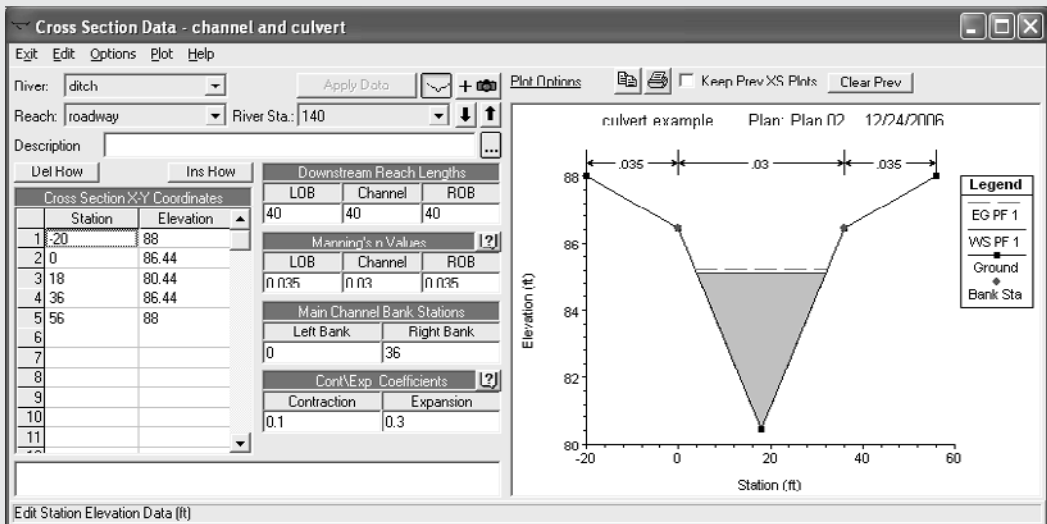


Figure E9-3(c)
Cross-section data for station 140.

Culvert Data Editor

Add Copy Delete ... Culvert ID: Culvert #1

Solution Criteria: Highest U.S. EG Rename ...

Shape: Box Span: 7 Rise: 3

Chart #: 58- Rectangular concrete

Scale #: 2 - Side tapered; More favorable edges

Distance to Upstrm XS: 4 Upstream Invert Elev.: 80

Culvert Length: 42 Downstream Invert Elev.: 79.58

Entrance Loss Coeff.: 0.5 # identical barrels: 1

Exit Loss Coeff.: 1

Manning's n for Top: 0.012

Manning's n for Bottom: 0.012

Depth to use Bottom n: 0

Depth Blocked: 0

Centerline Stations	
	Upstream Downstream
1	18. 18.
2	
3	
4	

OK Cancel Help

Select culvert to edit

Figure E9-3(d)
Culvert data for input.

A culvert is added at river station 75 using the options tab of the Bridge Culvert Data window within the Geometry Editor. Station 75 represents the center station between the upstream and downstream bounding stations of 100 and 50, respectively. The culvert is 42 ft long, and with the 4 ft upstream and downstream of the culvert ends to the nearest stations, the total distances add correctly (station 50 + 4 + 42 + 4 = station 100).

For the roadway deck, input the upstream and downstream high chord elevation of 86 for 50 ft left and right of the culvert (a length greater than what is needed to fully span the ditch). The elevation of 86 ft is the centerline elevation of the roadway for overtopping considerations. Leaving the low chord fields blank confirms to RAS that there are no openings in the embankment other than the culvert.

Save the geometry file once complete.

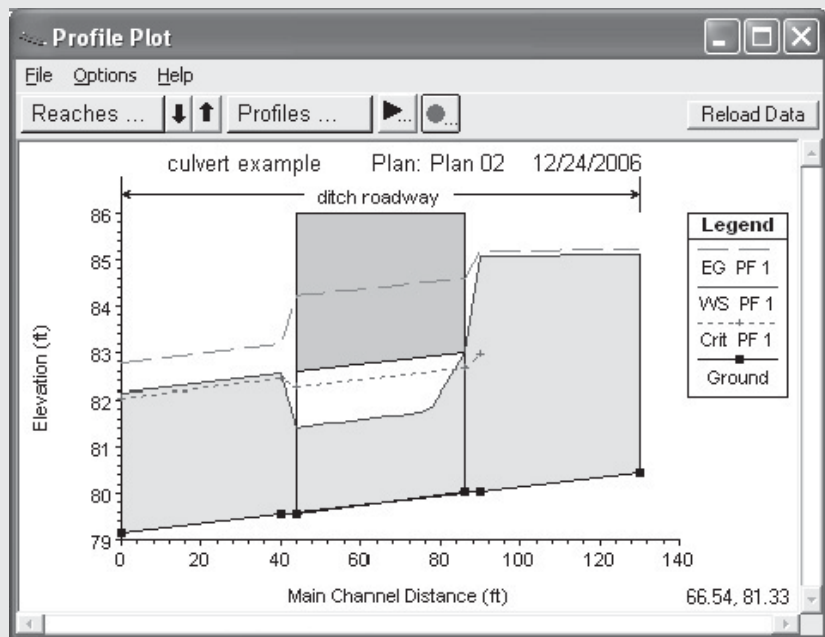
3. Create a new steady-flow data file by selecting the Enter/Edit Steady Flow Data shortcut button and then selecting: *File* → *New Flow Data*. Enter/Edit Number of Profiles is 1. River is *Ditch*, Reach is *Roadway*, and River Station is 140 (the upstream project cross section for this example). Enter in 172.8 cfs for Profile 1.

Next select Reach Boundary Conditions, the Normal Depth button, and enter a slope of 0.01 in the pop-up for the slope as the downstream slope for the normal depth calculation.

Save the steady-flow file as *inlet.f01* or another desired file name.

4. Select the Run shortcut button (“Running Man”) for a steady-flow simulation. Open a new plan by selecting the new geometry data file and new steady-flow file created in steps 2 and 3 above. Press Compute and view results such as the Cross Section plot, Detailed Output Table, Profile Plot, X-Y-Z Perspective Plot, and the Culvert Output under the detailed output shortcut. From the Culvert Output window, note that the culvert is under inlet control. A view of the Profile plot illustrates the satisfactory behavior of the culvert system [Fig. E9–3(e)].
5. Now that the analysis has been performed on the 7×3 culvert and the results examined satisfactorily, change the culvert size to 6×3 in the Culvert Data Editor window and examine the results as you did above. Notice that from the Culvert Output table and as viewed in the profile plot [Fig. E9–3(f)], weir flow over the roadway (i.e., roadway overtopping) is occurring. Refinement of the runs will be needed to accurately assess the true weir flow; however, since roadway overtopping was not permitted in the initial project parameters, a 6×3 box culvert will not work satisfactorily at this location.

Figure E9–3(e)
Profile plot of
 7×3 culvert
illustrating inlet-
control behavior.



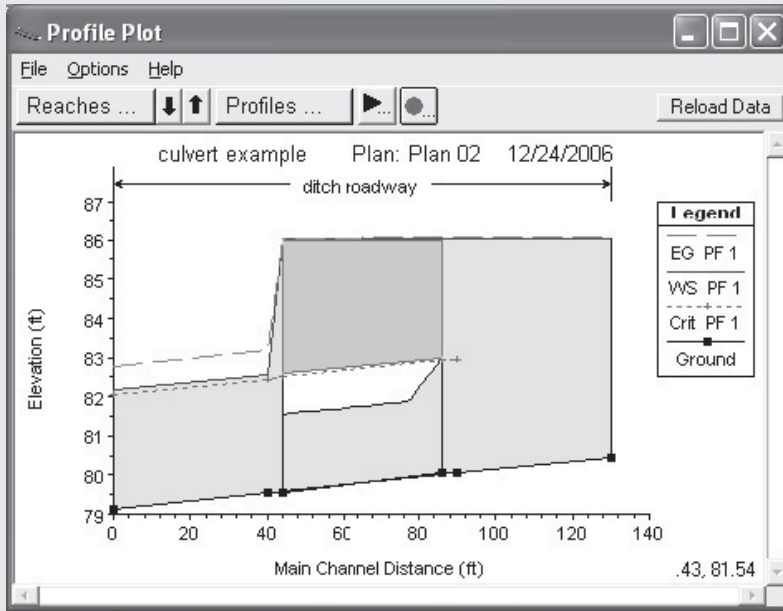


Figure E9-3(f)
Profile plot of
6 × 3 culvert
illustrating
roadway
overtopping
(weir flow over
roadway).

Detention Concepts and Applications

The basic premise for the use for detention basins within drainage collection systems is to reduce the peak discharges to predeveloped or preproject conditions. In floodplain management design, detention basins can have other uses, as will be discussed later within this chapter. Detention basins are commonly used in conjunction with other infrastructure improvements, such as with the design of new land developments and storm sewer and roadside ditch collection system improvements.

The development of an improvement project within an existing watershed can consist of not only the construction of new facilities on raw land, but also the alteration of existing impervious levels and drainage characteristics in previously developed areas. As such, new developments, channel modifications, roadway reconstructions, and other common civil works projects may lead to the creation of undesirable hydraulic impacts in the form of increased peak runoffs from various storm events. The identification, quantification, and mitigation of such impacts to receiving outlet channels or conduits and adjoining properties have become a primary focus in a typical project development life cycle. This is a main activity for design engineers and engineering hydrologists.

The term "impact" is used extensively in design as related to a particular project's development. By strict definition, all changes to the behavior of

9.4 DETENTION BASINS USED TO MITIGATE PROJECT IMPACTS

existing stormwater flows as routed from or through a project area could be deemed impacts. These impacts may be negative or positive. The alteration of an existing watershed's drainage characteristics, as caused by an urban development project, for example, often leads to increased flow rates to the receiving outlet or downstream collection system. In many cases, these increased flow rates equate to rises in open-channel water surface elevations or the surcharging of existing storm sewer systems, which in turn often directly relate to increased levels of surface flooding.

While the elimination or significant reduction of the increase of surface flooding is desired, not all increases in flow rates from a given project area negatively impact receiving outlet channels or closed conduit systems. This would be the case if an outlet system had sufficient capacity to facilitate these increased flows without any effect or alteration to levels of surface flooding.

Considering this discussion on various baseline parameters for the identification of negative impacts, the term *impact* actually implies *adverse impact*. An adverse impact would exist from a project development standpoint if the resultant increase in existing runoff flow rates caused by the alteration to the watershed characteristics directly resulted in increased surface flooding, erosion, or other such physically damaging phenomena. After an improvement project is designed, using either advanced or conventional means, the system should be analyzed to ascertain if any adverse impacts do indeed exist as a result of the new or proposed improvements. If these impacts do exist, then mitigation measures should be undertaken. Typically, the determination and mitigation of hydrologic and hydraulic impacts can entail a rather complex analysis utilizing complex tools. Simplified approaches are available in urbanized areas where storage coefficients or other such watershed parameters have been established to facilitate the development of stormwater management improvements. This may be the case, for example, if detention-basin requirements are simplified to a per-acre-of-detention-needed vs. the size of a given project area. As always, designers are encouraged to check local governing entities for such established criteria.

A common form of mitigation is the use of a detention basin within the project design. There are numerous detailed references on detention design, but the basic fundamental sizing of a detention basin for the purposes of mitigating increased runoff due to a particular project is based on the comparison of the existing to proposed discharge hydrograph at the outlet of the drainage collection system. The needed storage volume is determined from the area difference under the proposed vs. under the existing hydrograph, as illustrated in Figure 9–15. By providing this storage in the detention basin and properly sizing the outlet of the basin to govern the discharge to the desired predevelopment or preproject conditions, the negative effects or impacts of the given improvement project can be mitigated to mimic the existing-conditions peak flow rate, as can be seen in Figure 9–15. In other words, a properly designed detention basin can reduce the peak flow for the project proposed conditions to approximate the project existing conditions.

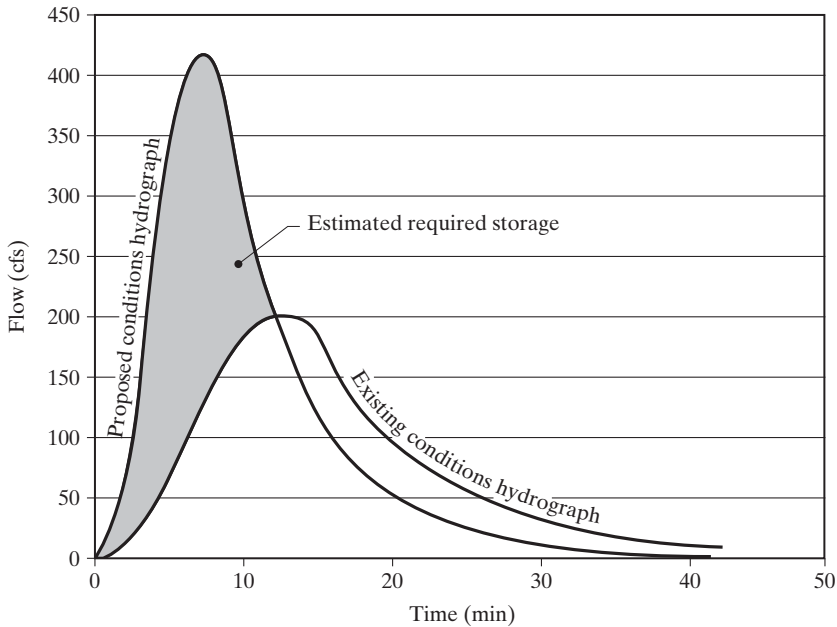


Figure 9-15
Existing and
proposed condi-
tions hydrographs
illustrating an
estimation of
required storage.

In this discussion and throughout this chapter, “existing conditions” are synonymous with “predevelopment conditions” and “preproject conditions.” Similarly, “proposed conditions” are synonymous with “postdevelopment conditions” and “postproject conditions.”

It should be noted that the volume of runoff, at times, is indeed greater in the postproject improvement conditions (i.e., the proposed conditions). This is a common phenomenon when the impervious cover of a project area is increased. By the use of a properly sized detention basin, as described above, the peak discharge of the proposed conditions is reduced to that of the existing conditions, and the additional runoff volume (stored in the basin and in the drainage collection system) is released over time.

Mitigation measures using detention basins are not easily employed in densely developed urban areas. In many cases, available land for detention basins either does not exist or is too costly. As is often the case, a complex balance of manipulating available storage within storm sewers, ditches, and even depressed curb-and-gutter streets in conjunction with adjusting the routing of flows within the storm drainage system is required to successfully mitigate adverse impacts resulting from the proposed improvements. For these reasons, complex modeling techniques using sophisticated computer software such as the EPA’s Storm Water Management Model (SWMM) are often required to properly analyze the project conditions, make an accurate determination of adverse impacts, and adequately design the needed mitigation measures, which may or may not employ a detention basin or basins (see Chapter 6).

Detention Basin Design

The actual design process of a detention basin is rather straightforward. As mentioned above, some communities already have predetermined parameters for detention basin sizing that are based upon percentages of change in imperviousness or other such criteria. Given the absence of such predetermined criteria for design purposes, the basic steps in the design of a detention basin are:

1. Develop the existing-conditions (preproject) runoff hydrograph for the project area.
2. After designing the drainage collection system or system improvements for the project area, develop the proposed-conditions (post-project) runoff hydrograph for the project area.
3. Comparing the existing- to proposed-conditions hydrographs, determine the detention volume needed to reduce the proposed peak discharge.
4. Size the detention outlet and basin to use the storage provided and to govern the discharge as desired.

The above steps are for commonly applied detention basins used in conjunction with drainage collection systems. This is known as “in-line” detention, whereby the detention basin is actually in line with the drainage collection system, typically at the utmost downstream end, as connected to the outfall or receiving channel or other drainage system. “Off-line” detention systems are commonly applied in floodplain management and typically involve side-flow weirs for connectivity to the drainage system. Detention basins used with drainage collection systems may indeed be off line, whereby the project area discharge conduit is restricted to govern the flow of discharge and a detention basin is connected to the collection system in a lateral sense; hence, it is an off-line basin. Off-line detention basins are discussed later in this chapter in conjunction with floodplain management design techniques.

Many features and appurtenances are commonly employed in detention basins. One such feature is the utilization of a flap gate or pinch valve at the outlet to the receiving outfall channel. Such a flap gate or similar device prevents backflow into the detention basin as the outfall channel rises during a storm event, thereby utilizing the storage available in the basin. In some cases, the provision of detention storage within the watershed is viewed as a positive feature regardless of whether the inflow comes from a project site or from backflow from the receiving outfall channel. The use of flap gates or pinch valves with detention basins is just one of many design decisions in project planning. Should such a gate or valve be used, the head loss through it is sometimes ignored, as modern gates and valves can be specially fitted or even custom made for each application, resulting in rather minor losses in their use.

DETENTION-BASIN DESIGN**EXAMPLE 9-4**

A 150-acre new land development is planned for a given area that is currently undeveloped, open grassland in good condition. Next to the development site is a natural open channel that will serve as the project outfall. A detention basin is to be constructed adjacent to the open-channel outfall, as shown in Figure E9-4(a), to reduce (i.e., mitigate) the 100-yr proposed (postproject or developed) condition peak discharge rate from the project area to approximate the 100-yr existing (preproject or undeveloped) condition peak discharge rate as required by the local governing authority. A flap-gate or pinch-valve outlet structure will be constructed in the basin outlet conduit to prevent backflow from the outfall channel back up into the detention basin (i.e., no reverse flow in the outlet conduit). Ignore the minor loss of the flap gate or pinch valve. SCS TR-20 methodology is to be used with the following data to develop the existing (preproject) and proposed (postproject) conditions hydrographs, and these hydrographs are to be used in the sizing of the basin and outlet to the receiving project outfall channel.

Existing (preproject) $T_c = 1.0$ hr

Proposed (postproject) $T_c = 0.45$ hr

Existing Condition SCS Curve No. = 61

Proposed Condition SCS Curve No. = 75

24-hr, 100-yr Rainfall Depth = 8.0 in.

Rainfall Distribution = SCS Type II, 24-hr

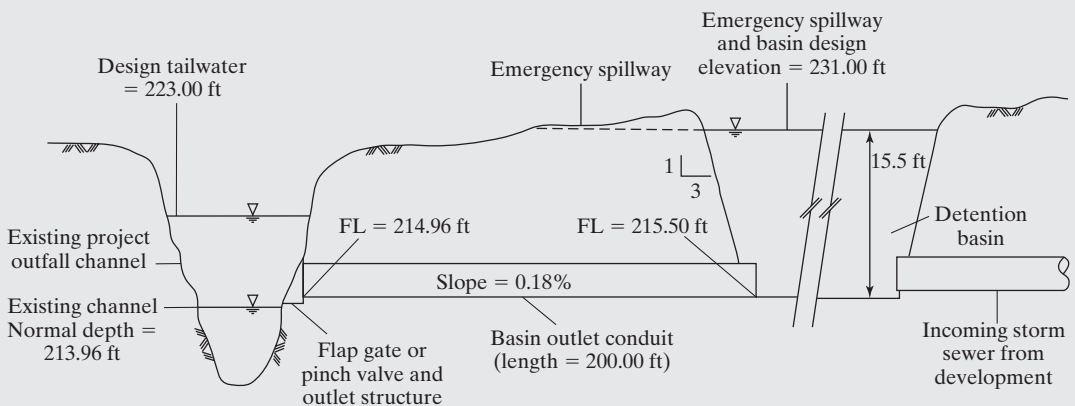


Figure E9-4a

Detention basin characteristics and design parameters.

SOLUTION

1. Develop the existing- and proposed-conditions runoff hydrographs using SCS TR-20 methodology. There are many means of achieving this. Chapter 2 includes a brief description of this methodology. As commonly applied, computer programs are readily available that quickly compute runoff hydrographs based on TR-20 methodology. One such program is the WinTR-55 Small Watershed Hydrology program, available from the Natural Resource and Conservation Service (NRCS; formerly the SCS), a branch of the USDA. WinTR-55 is a single-event rainfall-runoff watershed model that uses the TR-20 methodology for developing runoff hydrographs [see Fig. E9-4(b)]. The program can be downloaded directly from the USDA and NRCS website (see Appendix E). WinTR-55 computes peak discharges and provides a graphic plot of runoff hydrographs. Furthermore, WinTR-55 produces a TR-20 hydrograph text output file that can be exported to any spreadsheet for use in calculating required storage volumes or for other reasons in support of a given design.

After input of the above data for the existing and proposed conditions, WinTR-55 can be run to generate the hydrographs for export to a spreadsheet (but the tabular TR-20 hydrograph text

Figure E9-4(b)
Main window of WinTR-55 for existing- and proposed-conditions hydrograph generation.

WinTR-55 Main Window

File Options ProjectData GlobalData Run Help

WinTR-55 Small Watershed Hydrology

Project Identification Data

User: J Thompson State: Texas

Project: Detention Pond Example County: Harris

Subtitle: Execution Date: 5/21/2007

Sub-areas are expressed in:

☒ Acres ☐ Square Miles

Dimensionless Unit Hydrograph: <standard>

Storm Data Source: User-provided custom storm data

Rainfall Distribution Identifier: Type II

Sub-area Entry and Summary

Sub-area Name	Sub-area Description	Sub-area Flows to Reach/Outlet	Area (ac)	Weighted CN	Tc (hr)
Existing	Pre-Project Conditions	Outlet	150.00	61	1.000
Proposed	Post-Project Conditions	Outlet	150.00	75	0.450

Note that the Project Area is really only 150 Ac.
Both hydrographs are being generated in the same run

Project Area: 300 (ac)

File: C:\Documents and Settings\thompsonj\Application Data\WinTR-55\detentor 5/21/2007 3:10 PM

data can be obtained from WinTR-55 and the data can be plotted and utilized by hand methods equally as well!).

2. Within an electronic spreadsheet (or by hand methods), overlay the existing and proposed hydrographs and visualize the impacts of the proposed (postproject) conditions. Measure the area under the proposed-condition curve compared to the existing-condition curve to estimate the storage volume required, as illustrated in Figure E9-4(c). Also, note the maximum existing-condition discharge or runoff from this figure (or taken directly from the WinTR-55 output).
3. Size the outfall pipe for a maximum discharge equal to the existing peak flow at the maximum allowable head differential between the tailwater elevation at the outfall and the maximum allowable water surface elevation in the detention basin. A commonly used equation for the head loss through a culvert flowing full is

$$H_t = (1 + Ke + 29n^2L/R^{1.33})V^2/2g.$$

However, this requires solving iteratively and calculating R and v repeatedly. For round pipe conduit (the type of conduit that will be used for the basin outlet conduit), this equation can be simplified to

$$H_t = (1 + Ke + 183n^2L/d^{1.33})0.81Q^2/(gd^4),$$

where

H_t = allowable head differential = 8.0 ft (231.00 – 223.00),

Ke = entrance loss coefficient = 0.5,

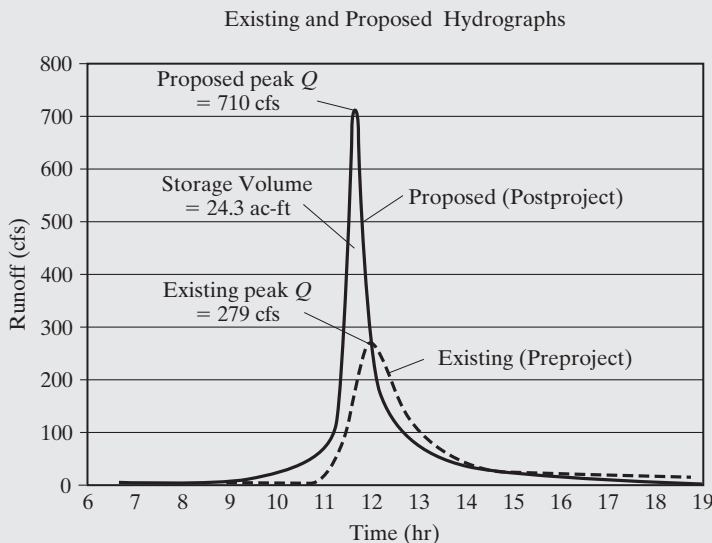


Figure E9-4(c)
Hydrographs for existing and proposed conditions developed from TR-55 output data.

n = Manning's roughness coefficient = 0.024,
 L = discharge conduit length = 200 ft,
 R = hydraulic radius,
 v = velocity in conduit,
 d = diameter of discharge conduit,
 Q = design discharge rate = existing peak flow = 279 cfs,
 g = acceleration due to gravity = 32.2 ft/s².

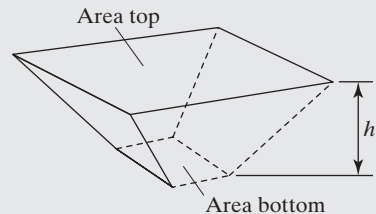
In the latter equation, the only unknown is d , and only this one equation need be solved iteratively, which reduces the calculation effort. By iterative trial and error, the discharge diameter is 5.48 ft. Round up to the next normal standard conduit size (in 0.5-ft increments); use standard conduit size of 5.5-ft (66-in.) diameter.

4. Determine the approximate dimensions of the detention basin, assuming the basin to be square with side slopes of 3:1, as shown in Figure E9-4(a). From the figure, the depth of the basin is based on the flow line of the discharge pipe and the design water surface elevation of the basin at the top of the emergency spillway, which results in a depth of 15.5 ft (231.00 – 215.50). The required volume was previously determined in step 2. With the depth and volume known, the equation for a pyramidal frustum can be used to determine the dimensions:

$$V = \frac{1}{3}h[A_{\text{top}} + A_{\text{bottom}} + \text{Sqrt}(A_{\text{top}} \times A_{\text{bottom}})]$$

where

V = volume,
 h = depth of basin,
 A = area.



The resulting dimensions are top side lengths of 306.44 ft and bottom side lengths of 213.44 ft. However, since the basin would be constructed holding depth, side slopes, and top side lengths constant, and we are starting with an estimate of the required volume, we will use top side lengths of 310 ft (slightly larger than the calculated 306.44 ft) with all other depth (15.5 ft) and slope (3:1) information the same. Should this prove inadequate in size in terms of the storage provided, we can increase the top side lengths.

5. With the discharge conduit sized and the dimensions of the basin established, develop a stage-storage-discharge relationship.

This can also be done in conjunction with developing data for a storage-indication curve that will be used in the routing. Each stage in the table will have a specified amount of storage, S (based on the geometry of the basin determined in step 5), a specified discharge from the basin, Q (based on the differential head between the tailwater and stage level in the basin for the conduit sized in step 3), and a value for $2S/\Delta t + Q$ based on the Q and S for that stage [see Table E9-4(a)]. This table will then be used in the routing procedure.

A note of caution at this point about terms: Chapter 4 gives detailed descriptions of the routing procedures used herein, and the terms used in the design can become confusing if not managed. We have an inflow rate into the basin and an outflow rate out of the basin, all working integrally with the change in storage in the basin itself. The term “discharge” is used to refer to the outflow leaving the basin to the outfall channel, and the term “ Q ” is used in the tables and figure that follow to refer to this discharge rate. The inflow is referred to by “ I ” as utilized in Chapter 4.

In the creation of the table in step 5 above, the elevation data are simply tabulated from the flow line of the basin to the emergency spillway elevation, using an increment of tabulation that will provide sufficient detail to determine the behavior of the basin. Experience plays a role in making the determination of the increments used. In this case, 35 incremental steps in terms of basin elevation were chosen to analyze the basin performance. Once the elevations are tabulated, then the stage, storage, and basin pool top area are calculated for each elevation increment. The discharge is then calculated based upon the equations in step 3 above. But note that discharge from the basin does not occur until after the basin elevation of 223.00 ft is exceeded, since the pool level in the basin must exceed the water surface elevation in the outfall channel in order for discharge to occur out of the basin. In addition, it should be noted that the flap-gate or pinch-valve feature at the end of the outlet conduit is keeping inflow from occurring back into the basin from the outfall channel, which would consume valuable storage in the basin. Should such a flap gate or pinch valve not be included in the design, the storage needed would have to be above elevation 223.00 ft (the design water surface elevation in the outfall channel), unless governing local criteria dictate otherwise.

Finally, Table E9-4(a) includes a column at the far right that will be used for the routing computations included in the next step. In order to complete this calculation, the time step to be used in the routing needs to be determined (see Chapter 4 for a detailed

explanation of this procedure and the equations utilized). A very small time step, of the order of, say, 1 minute, would produce a very detailed routing tabulation that might not produce any additional benefit. On the other hand, too great a time step, say, 1 hour, would not produce enough accuracy in the routing

Table E9-4(a). Table Relating Storage and Discharge for Basin

Elevation (ft)	Stage (ft)	Storage (cu ft)	Storage (ac-ft)	Area Top (sq ft)	Discharge, Q (cfs)	$2S/\Delta t + Q$ (cfs)
215.50	0.00	0	0.0	47089	0.00	0.00
215.96	0.46	21739	0.5	48284	0.00	48.31
216.41	0.91	44026	1.0	49493	0.00	97.83
216.87	1.37	66867	1.5	50718	0.00	148.59
217.32	1.82	90270	2.1	51957	0.00	200.60
217.78	2.28	114242	2.6	53212	0.00	253.87
218.24	2.74	138789	3.2	54481	0.00	308.42
218.69	3.19	163918	3.8	55765	0.00	364.26
219.15	3.65	189636	4.4	57065	0.00	421.41
219.60	4.10	215950	5.0	58379	0.00	479.89
220.06	4.56	242867	5.6	59708	0.00	539.70
220.51	5.01	270393	6.2	61053	0.00	600.87
220.97	5.47	298535	6.9	62412	0.00	663.41
221.43	5.93	327300	7.5	63786	0.00	727.33
221.88	6.38	356695	8.2	65175	0.00	792.65
222.34	6.84	386726	8.9	66579	0.00	859.39
222.79	7.29	417401	9.6	67998	0.00	927.56
223.25	7.75	448727	10.3	69432	49.69	1046.86
223.71	8.21	480710	11.0	70881	83.49	1151.73
224.16	8.66	513356	11.8	72345	107.11	1247.90
224.62	9.12	546674	12.5	73824	126.39	1341.22
225.07	9.57	580669	13.3	75318	143.09	1433.47
225.53	10.03	615348	14.1	76827	158.04	1525.48
225.99	10.49	650719	14.9	78351	171.69	1617.74
226.44	10.94	686788	15.8	79889	184.34	1710.53
226.90	11.40	723561	16.6	81443	196.17	1804.08
227.35	11.85	761047	17.5	83012	207.33	1898.54
227.81	12.31	799251	18.3	84595	217.91	1994.02
228.26	12.76	838180	19.2	86194	228.01	2090.63
228.72	13.22	877842	20.2	87808	237.67	2188.43
229.18	13.68	918243	21.1	89436	246.96	2287.50
229.63	14.13	959389	22.0	91080	255.91	2387.89
230.09	14.59	1001288	23.0	92738	264.56	2489.65
230.54	15.04	1043947	24.0	94412	272.94	2592.82
231.00	15.50	1087372	25.0	96100	281.06	2697.45

computation. Again, experience comes into play, and for this example 15 minutes is determined to be a suitable time step. Therefore, $\Delta t = 15$ minutes (900 sec), and the calculations relating storage, Δt , and discharge are made to complete Table 9-4(a).

The storage-indication curve as shown in Figure E9-4(d) is then produced and will be used in step 6 to complete the basin flood routing. This curve is a simple graphic relating the $2S/\Delta t + Q$ value to the discharge, Q (see Chapter 4).

6. Route the proposed hydrograph thru the detention basin and outfall conduit to determine if the peak outflow and the size of the basin are adequate, using the data developed in the previous steps and the routing procedure described earlier. In this example, there is no reverse flow into the basin from the outfall channel due to the flap-gate or pinch-valve structure, and outflow from the basin into the outfall channel does not occur until the elevation of the water in the basin is greater than the tailwater elevation in the outfall channel. Use the table developed in step 5 to determine the outflow and stage at the end of each routing step. Check the resulting routing to determine if the maximum stage or discharge exceeds design parameters, or if the basin was oversized and can be reduced in size. If either of the design parameters was exceeded, the basin needs to be resized to avoid this.

The following steps can be taken to follow Table 9-4(b):

- (a) Tabulate the time steps in hours and minutes based upon the determined time step, Δt , in step 5.

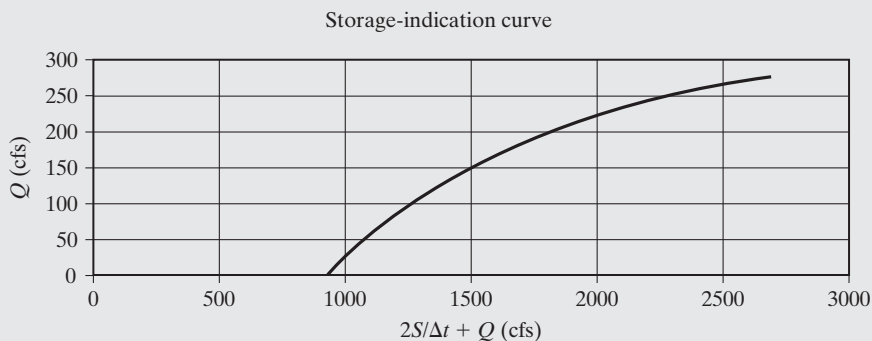


Figure E9-4(d)

Storage-indication curve used for basin routing.

- (b) Determine the inflow, I_{n+1} , from the proposed-conditions hydrograph or from the output table of the WinTr-55 run above.
- (c) Calculate $(I_n + I_{n+1})$ by simply summing the inflow at a given time step with the inflow from the previous time step.
- (d) Refer to the detailed routing description in Chapter 4, utilizing Equation (4-13) as restated below.

$$(I_n + I_{n+1}) + (2S_n/\Delta t - Q_n) = (2S_{n+1}/\Delta t + Q_{n+1}).$$

Note that at the first tabulated time step of 6.20 hr, all inflow, discharge, stage, and storage variables are equal to zero. At time 6.45 hr, $(2S_n/\Delta t - Q_n) = 0$ because S_n and $Q_n = 0$ (the storage and discharge at the previous time step, 6.20 hr). Therefore, solving the above equation yields $(2S_{n+1}/\Delta t + Q_{n+1}) = 0.06$ cfs. Q_{n+1} is then determined directly from the storage-indication curve, interpolating between the data points as prepared in step 5. Subsequent time steps are calculated in the same fashion.

- (e) Next, the storage is computed for each time step by solving S_{n+1} . Continuing with the step at 6.45 hr,

$$(2S_{n+1}/\Delta t + Q_{n+1}) = 0.06 \text{ cfs.}$$

$$Q_{n+1} = 0,$$

$$\Delta t = 15 \text{ min} = 900 \text{ sec.}$$

Therefore,

$$S_{n+1} = \text{storage} = 27 \text{ ft}^3.$$

The calculation is repeated for every time step.

- (f) For every known storage value, the stage is determined via interpolation from Table E9-4(a) created in step 5.

The results tabulated in Table E9-4(b) demonstrate that the basin acts efficiently. The maximum stage elevation of 15.5 ft is not exceeded; however, the basin is almost fully utilized (maximum stage elevation of 15.02 ft). Notice that no outflow occurs until the minimum stage of 7.50 ft is achieved (223.00 ft – 215.50 ft), and in actuality the basin would drain out as the outfall channel flood water surface elevation recedes to its normal base flow depth. The inflow and outflow for the basin plot similarly to that shown in Figure E9-4(c). Notice the maximum discharge from the basin is 272.65 cfs, which is less than the existing-conditions peak discharge of 279 cfs, thereby indicating a mitigated proposed condition.

Table E9-4(b). Detention Basin Routing

Time (hr)	Time (min)	I_{n+1} (cfs)	$(I_n + I_{n+1})$ (cfs)	$(2S_n/\Delta t - Q_n)$ (cfs)	$(2S_{n+1}/\Delta t + Q_{n+1})$ (cfs)	Q_{n+1} (cfs)	Storage (cu ft)	Stage (ft)
6.20	372	0.00	0.00	0.00	0.00	0.00	0	0.00
6.45	387	0.06	0.06	0.00	0.06	0.00	27	0.00
6.70	402	0.39	0.45	0.06	0.51	0.00	230	0.00
6.95	417	0.86	1.25	0.51	1.76	0.00	792	0.00
7.20	432	1.39	2.25	1.76	4.01	0.00	1805	0.00
7.45	447	1.95	3.34	4.01	7.35	0.00	3308	0.00
7.70	462	2.53	4.48	7.35	11.83	0.00	5324	0.00
7.95	477	3.14	5.67	11.83	17.50	0.00	7875	0.00
8.20	492	3.77	6.91	17.50	24.41	0.00	10985	0.00
8.45	507	4.63	8.40	24.41	32.81	0.00	14765	0.00
8.70	522	5.78	10.41	32.81	43.22	0.00	19449	0.00
8.95	537	7.11	12.89	43.22	56.11	0.00	25250	0.00
9.20	552	8.59	15.70	56.11	71.81	0.00	32315	0.00
9.45	567	9.85	18.44	71.81	90.25	0.00	40613	0.00
9.70	582	10.94	20.79	90.25	111.04	0.00	49968	0.00
9.95	597	12.65	23.59	111.04	134.63	0.00	60584	0.00
10.20	612	15.12	27.77	134.63	162.40	0.00	73080	0.00
10.45	627	18.40	33.52	162.40	195.92	0.00	88164	0.00
10.70	642	22.60	41.00	195.92	236.92	0.00	106614	0.00
10.95	657	28.35	50.95	236.92	287.87	0.00	129542	0.00
11.20	672	36.06	64.41	287.87	352.28	0.00	158526	0.00
11.45	687	48.84	84.90	352.28	437.18	0.00	196731	0.00
11.70	702	88.96	137.80	437.18	574.98	0.00	258741	0.00
11.95	717	392.87	481.83	574.98	1056.81	52.82	451796	7.79
12.20	732	683.58	1076.45	951.17	2027.62	221.42	812790	12.47
12.45	747	320.93	1004.51	1584.78	2589.29	272.65	1042488	15.02
12.70	762	159.87	480.80	2043.99	2524.79	267.41	1015821	14.74
12.95	777	99.56	259.43	1989.97	2249.40	243.39	902705	13.50
13.20	792	74.79	174.35	1762.62	1936.97	211.59	776421	12.04
13.45	807	61.85	136.64	1513.79	1650.43	176.15	663426	10.65
13.70	822	53.83	115.68	1298.13	1413.81	139.53	573426	9.47
13.95	837	47.51	101.34	1134.75	1236.09	104.21	509346	8.60
14.20	852	42.33	89.84	1027.67	1117.51	72.46	470273	8.06
14.45	867	38.81	81.14	972.59	1053.73	51.90	450824	7.78

Note: Once the storage is clearly declining, the routing may be stopped, as the maximum stage will have been reached (in this case, 15.02 ft).

Management Applications

There are countless scenarios for design applications in modern floodplain management. Typically, a main objective of such applications is flood damage reduction. In a simple sense, this involves the reduction of flood levels

9.5 FLOODPLAIN MANAGEMENT DESIGN ISSUES

for a given storm event, such that properties and structures are not negatively affected by high flood waters. In many cases, the reduction of flood waters involves the protection of human life as well. In other applications of floodplain management, designs are often employed to provide prevention of erosion at bridge structures and the protection of stream banks and other such physical features.

Flood damage reduction is a whole area of risk and predictability analyses. In all civilizations throughout history, in all locations around the world, flood damage reduction in the form of prevention has been a key factor in the development of human communities. It is common for communities to be developed next to or near existing natural channels (i.e., rivers, streams, and so on) for reasons of quality of life, transportation, and trade. Riverine flooding of properties and structures occurs for three fundamental reasons:

1. The properties and structures are in the floodplain of a given storm event. In other words, they are simply too close to the channel or within the channel's associated natural flood-prone areas;
2. The channel's flood level for a given storm event has risen due to increased development of the surrounding community, such that more peak runoff discharges result in higher water surface elevations in the channel; or,
3. A combination of the above two.

Other flooding scenarios can arise from poor or inadequate stormwater infrastructure. Regardless of the reasons, flood damage reduction is sought to reduce repetitive flooding losses and the associated trauma.

There are many ways to employ flood damage reduction techniques. Waterproofing is a simple means of protecting properties and structures, as with the use of waterproof flood doors, for example. Yet, it is commonly deemed necessary to reduce the floodwaters of a given channel, either a natural or man-made channel, via designed improvement schemes. Two fundamental schemes commonly employed are 1) channel improvements, and 2) detention. Channel improvement is simply increasing the conveyance characteristics of a given channel reach. Obviously, increasing the cross-sectional area of a given channel reach, while keeping the same flow, will reduce the water surface elevation in the channel and increase the storage capacity of the channel reach. Figure 9–16 gives a simple relationship of channel improvements as discussed herein.

At first glance, it appears ideal to simply increase a given channel cross section (i.e., employ channel improvements) in order to reduce the water surface elevation for a given storm event; however, related issues can have negative effects on the surrounding environs if not addressed correctly. When the water surface elevation is reduced due to channel improvements, then the effect of storage change must be considered. Looking at Figure 9–16, the reduction in water surface elevation, given the same flow rate from a

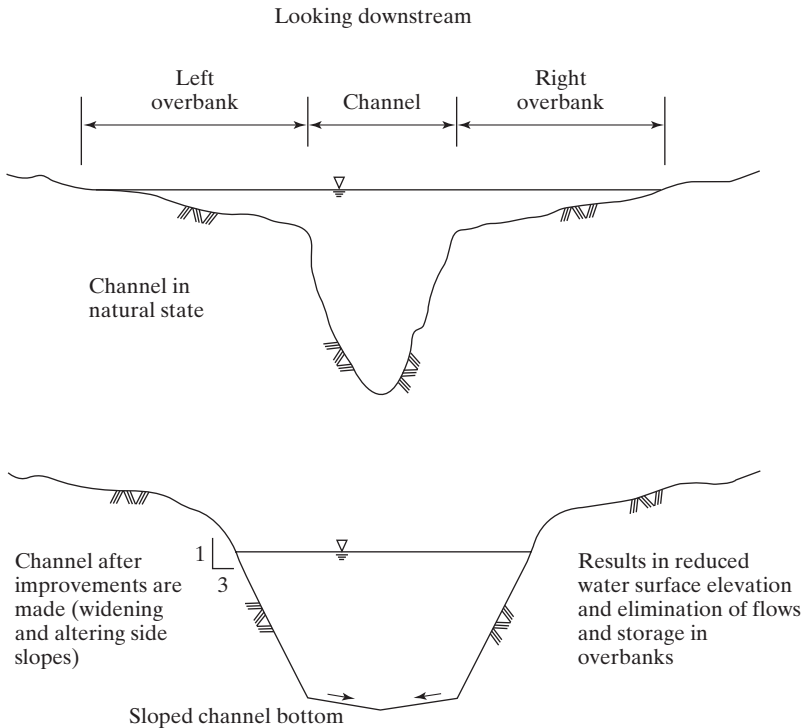


Figure 9-16
Channel improvements made to a natural channel.

particular storm event, might appear a very desirable outcome. However, consider the changes in storage. Prior to the channel improvements, there was a given volume of water in the left and right overbanks. With the water surface elevation reduced after channel improvements, this storage is no longer utilized, and the volume of water previously stored in the overbank regions is now routed downstream within the channel. In areas of large floodplains where the storage within the said overbank regions is significant, the volume of flood water routed downstream after channel improvements have been made can be equally significant, resulting in downstream increases in water surface elevation. In effect, the improvement (i.e., enlargement) of a given channel cross section for the lowering of water surface elevations along a river or channel reach can directly result in the routing of increased flows downstream, which in turn can result in increased water surface elevations downstream. This whole concept of channel improvements in relation to conveyance, storage, and routing is fundamental to the desired outcome of flood damage reduction without inducing negative affects along the river or channel system.

Another commonly employed means of lowering water surface elevations for a given storm event in a river system is detention. The behavior and characteristics are similar to those discussed above dealing with reduction

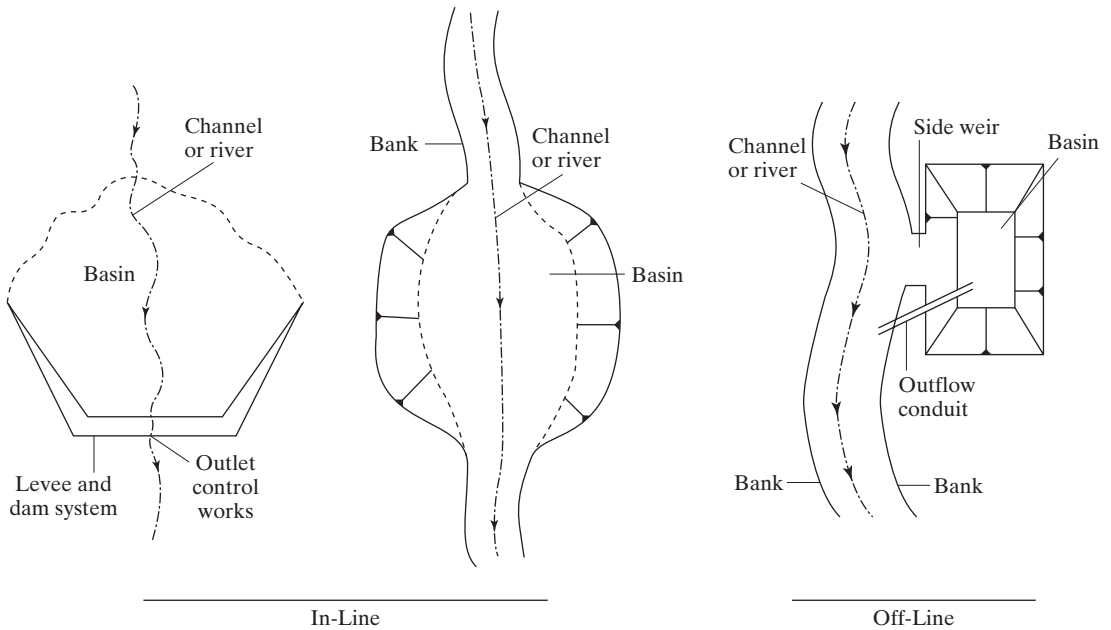


Figure 9-17
Open-channel detention basin types.

of peak discharges. In essence, additional storage volume is added to the channel system, and this storage, if properly planned and designed, can aid in the reduction of water surface elevations for a given storm event (i.e., provide some flood damage reduction potential). If not properly planned and designed, a detention basin may add little to no positive effect in flood damage reduction.

Detention basin types—in-line and off-line—were described earlier. Figure 9-17 illustrates commonly applied in-line and off-line detention basin designs. Example 9-4 describes a relatively small in-line basin at the end of a drainage collection system. In-line basins can be effectively constructed by simply widening a channel to provide more storage at a planned location; however, in large-scale open-channel or river projects, in-line basins often need to be substantial in size to provide the desired effects. A large dam and reservoir in a river system is an example of a very large in-line basin, although technically it is a *retention* basin, as the basin itself—in this case the reservoir—actually is intended to *retain* water at all times. In-line detention basins for common larger flood control projects usually involve a large levee-and-dam system integrated in-line with a natural channel system. As a flood event occurs, the flood waters are detained behind the levee-and-dam system, and an outlet structure or structures control the release of flood waters downstream. This is similar to the applied concepts used in Example 9-4,

dealing with an in-line detention basin for a development project—just on a much larger scale.

The second fundamental type of detention basin commonly utilized, an off-line basin, is adjacent to the channel or river and is hydraulically connected to the channel by means of a side weir or conduit or both. An outflow conduit is typically provided from the bottom of the off-line basin back to the channel to provide stored flow in the basin to reenter the channel as the flood waters recede. A flap gate is commonly provided in the outflow conduit to prevent flows from entering the conduit from the channel. The intent is to provide a storage facility of an adequate size to help reduce the peak of the flood hydrograph in the channel or river by setting the side weir (typically a broad-crest weir) at the right elevation, such that flows enter the basin at the correct time and flood-wave height in the channel. If the weir is not set at the correct height (e.g., is set too low), then the basin can fill up prior to the peak of the flood wave propagating down the channel, resulting in all of the volume of the basin being utilized before the storage is needed to reduce the peak of the channel flood wave. It is common to use terms like “cutting off the top of the hydrograph” when discussing the propagation of flood waves down a given channel in relation to the use of off-line detention basins. What this means is that the side weir is set at an appropriate elevation and sized correctly with the basin, such that as the hydrograph at the river station adjacent to the basin rises in terms of flow relative to time, the hydrograph is “cut off” at a planned level, as the flow enters the basin via the established side weir. The end result is an off-line basin that provides storage at the desired time so as to reduce the peak discharge in the channel, thus reducing the peak water surface elevation.

Flood Damage Reduction Design Using HEC-HMS and HEC-RAS

A fundamental hydrologic and hydraulic analysis and design process intended to address the impacts of urbanization on a watershed is the development of flood damage reduction (FDR) projects. FDR can take many forms, ranging from structural solutions, such as channelization and detention to reduce water surface elevations, to nonstructural solutions, such as buyouts of homes to reduce damages caused by the high water surface elevations. Combinations of structural and nonstructural activities are often the most cost-effective solutions. Simply employing channel improvements to reduce water surface elevations and thereby alleviate flooding in one particular location or area can exacerbate flooding in other areas. This is generally due to the routing of previously stored flood waters downstream, as described previously within this chapter and in Chapter 12. An important process in an overall FDR analysis and design is to fully understand the behavior and response of the watershed, given the existing conditions and any planned improvements to reduce flood damage.

There are many tools for developing reductions in water surface elevations that yield flood damage reduction benefits. Some of the most readily

available and commonly used are the HEC-HMS and HEC-RAS computer programs introduced in previous chapters. HEC-HMS can be readily used to evaluate the hydrology of a given watershed to examine the affects of development and urbanization. HEC-RAS can be used to efficiently model the behavior of a given channel or river system and then evaluate several design scenarios to provide reductions in water surface elevations. For detention basin design, HEC-RAS along with HEC-HMS can be utilized for full routing and the development of storage–discharge relationships, or HEC-RAS can be used in the unsteady simulation mode, whereby a known hydrograph is routed along a channel system in conjunction with a detention basin.

As the best way to illustrate the process described above, the following example is presented in stages that build upon one another in the development of an actual FDR design scenario.

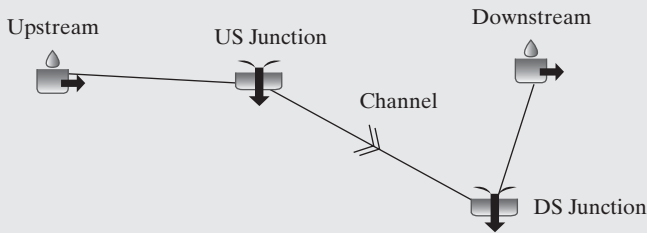
EXAMPLE 9–5**FLOOD DAMAGE REDUCTION DESIGN AND MITIGATION**

In this example, we will observe the impact of urbanization on a simple watershed and develop a structural flood damage reduction (FDR) project to mitigate the impact of the urbanization. The site will be modeled at several stages of development, ranging from Preexisting to Proposed conditions, with a structural FDR project in place.

There are four main stages of this example, as outlined below. The user of this example may perform any or all stages. At the conclusion of each stage, sufficient information is provided to initiate the next stage.

- Stage 1: Preexisting-conditions HEC-HMS and HEC-RAS analysis to give a baseline of how the watershed responded prior to any human development.
- Stage 2: Existing-conditions HEC-HMS and HEC-RAS analysis to understand how the watershed is performing after urbanization has taken the watershed to existing development conditions.
- Stage 3: Proposed-conditions HEC-HMS and HEC-RAS analysis and design to develop channel improvements to reduce flooding in the overbank regions of the channel for flood damage reduction (FDR) purposes.
- Stage 4: Mitigated Proposed conditions with an off-line detention basin HEC-RAS unsteady analysis to develop a design of a detention basin to mitigate downstream impacts caused by the channel improvements implemented in stage 3 above.

This example is completely developed using HEC-HMS 3.1.0 and HEC-RAS 3.1.3 and assumes some working familiarity with the software.

**Figure E9-5(a)**

Schematic of the watershed and its connectivity.

Both programs have very good built-in help functions that should be of assistance in case anything below is not completely clear. (HEC-HMS and HEC-RAS are available from <http://www.hec.usace.army.mil>.)

All of the examples use the same watershed with the same basic connectivity. The physical layout, connectivity, and parameters of a watershed are entered into a Basin Model in HEC-HMS. The Upstream subbasin flows to an upstream junction (US Junction) and then through a reach called Channel to a downstream junction (DS Junction), where its flow combines with flow from the Downstream subbasin, as shown in Figure E9-5(a). The elements of HEC-HMS were covered in detail in Chapter 5. Each item can be selected and its parameters can then be assigned to it or modified.

For the purpose of this example, the same synthetic storm with a 1% exceedance probability (i.e., a 100-yr synthetic design storm) will be used for all four stages. The rainfall pattern is input into the Meteorologic Model of HEC-HMS using the Frequency Storm precipitation method with the data shown in Table E9-5(a). The storm duration is 24 hr (1 day), the Intensity Position is at 50% (the peak rainfall occurring halfway

Table E9-5(a). Frequency Storm
Precipitation Method Depths for
HEC-HMS Meteorologic Model

Duration	Depth (in.)
5 min	1.2
15 min	2.1
1 hr	4.3
2 hr	5.7
3 hr	6.7
6 hr	8.9
12 hr	10.8
24 hr	13.2

through the duration of the storm), and the maximum Intensity Duration is 5 minutes.

Stage 1: Preexisting Conditions

Preexisting conditions describe the watershed before urban development. In this case, there are two subbasins, each 2.0 mi² in area, and with the same hydrologic characteristics represented by the following Clark Unit Hydrograph parameters: Time of Concentration, $T_c = 1.0$ hr; and Storage Coefficient, $R = 5.0$ hrs.

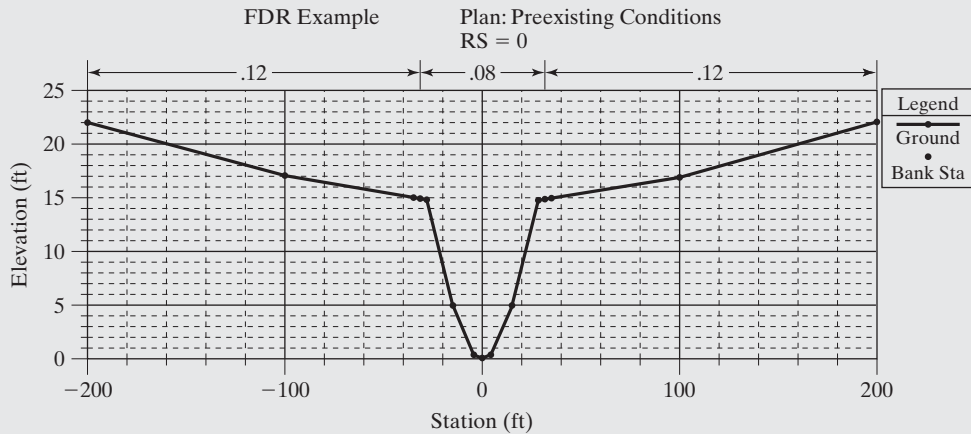
In HEC-HMS, the rainfall data is entered in a Meteorologic Model, with a Frequency Storm precipitation method. Once the Meteorologic Model has been created, it will need to be specifically authorized to work with a given Basin Model. Otherwise, an error message in HEC-HMS will remind you to address the issue. This example uses a 5-minute time step for the HEC-HMS models. The time-step size and the simulation start/end dates for the model are set using the Control Specifications.

The Channel will be modeled using the Modified-Puls method, which determines the outflow from a channel segment based on the inflow and a relationship between channel storage and outflow. The storage–discharge relationship (sometimes called “S vs Q” or “SvsQ” or “SVSQ”) is typically determined from a hydraulic model (i.e., HEC-RAS) as a table of volume in the system for a series of different flow profiles. Chapter 4 in this text and Chapter 8 of the HEC-HMS Technical Reference Manual (included with the software download) have brief descriptions of the Modified-Puls method and how to develop SvsQ relationships. Only a brief outline of the detailed procedures is given within this example.

In this case, the Preexisting-conditions channel is a natural channel with steep banks and a fairly flat floodplain in the overbanks. The overall topography of the project area can be classified as very flat. The channel is 2.4 miles (12,672 ft) long, has a uniform channel slope of 0.00075 ft/ft, and has the station/elevation data presented in Table E9–5(b). Figure E9–5(b) shows the downstream end of the channel (River Station 0, RS 0, or RS = 0), along with the roughness values of the channel and the two overbanks. The upstream end is an exact copy at RS 2.4 that has had the

Table E9–5(b). Example Cross-Section Station/Elevation Data for Channel at Furthest Downstream End (RS = 0); All Station and Elevation Data in Feet

Station	Elevation	Station	Elevation	Station	Elevation
–200	22	–15	5	29	14.8
–100	17	–5	0.5	32	15
–34	15.1	0	0	34	15.1
–32	15	5	0.5	100	17
–29	14.8	15	5	200	22

**Figure E9-5(b)**

Channel cross section at furthest downstream end (RS = 0).

elevation adjusted up 9.51 ft for all cross-section points and has left overbank (LOB), channel, and right overbank (ROB) downstream reach lengths equal to the channel length (i.e., there are no bends in the channel). The left and right bank stations are at -29 ft and 29 ft, respectively.

SvsQ relationships can be determined several ways. One method is to take the flow profiles from various hypothetical storms (e.g., 2-yr, 5-yr, 10-yr). This method produces good SvsQ curves, and as a side benefit, when the SvsQ iteration process is complete, all of the various storms have also been analyzed and the SvsQ for each will be internally consistent. A second method, used in this example, uses the peak flows from the 100-yr storm and several multipliers of the peak flow. These flows are entered into the HEC-RAS steady-flow file, which is then run, calculating the storage volumes. The multipliers used herein are 0.01, 0.05, 0.1, 0.2, 0.4, 0.6, 0.8, 0.9, 1.0, 1.1, and 1.25. This method with the multipliers does not always correctly model storage for other storms, as the SvsQ relationship can be impacted by when, in the curve, the low chord of a bridge is submerged. For this example, since we are using only one design storm in our analysis and we are modeling a straight uniform channel, this method is sufficient.

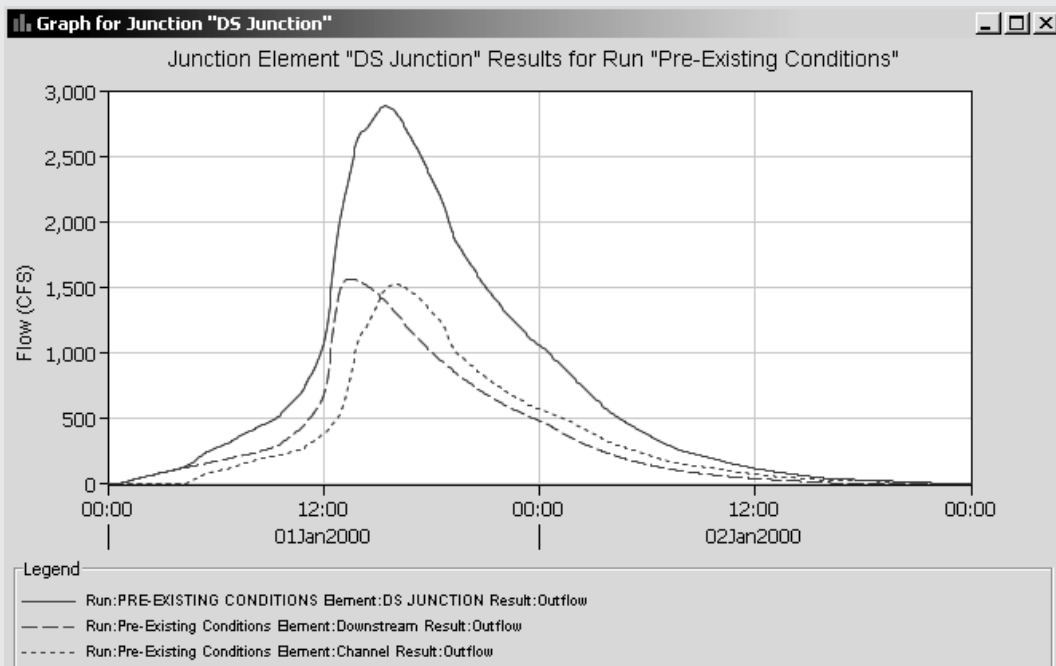
At this point, initial SvsQ values are determined using the HEC-RAS model output with an estimated 100-yr peak flow (3000 cfs is a good first estimate), and SvsQ iterations using the multipliers of the peak flow should be performed. In the SvsQ iterations, the SvsQ values from HEC-RAS are pasted into HEC-HMS, a new peak flow is found, and that new peak flow and its multipliers are used in HEC-RAS to get a new SvsQ relationship. This procedure continues until the peak flow stops changing within a suitable tolerance (usually within a few percent of the peak). At

Table E9-5(c). Storage–Discharge Relationship for Channel in Preexisting-Conditions HEC-HMS Model

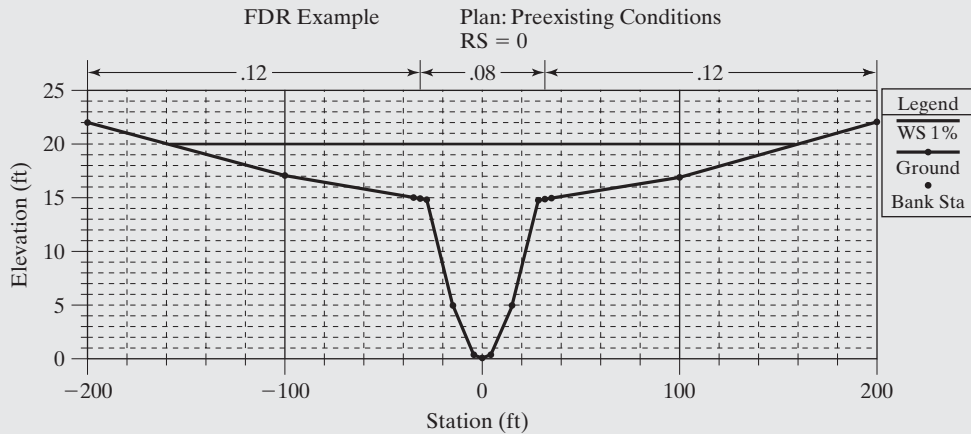
HEC-RAS Profile	Storage (ac-ft)	Discharge (cfs)
N/A	0.000	0.000
0.01 × 100-yr	11.190	28.807
0.05 × 100-yr	35.850	144.030
0.1 × 100-yr	58.770	288.070
0.2 × 100-yr	97.270	576.130
0.4 × 100-yr	161.450	1152.300
0.6 × 100-yr	262.270	1728.400
0.8 × 100-yr	372.110	2304.500
0.9 × 100-yr	425.550	2592.600
1.0 × 100-yr	478.290	2880.700
1.1 × 100-yr	529.920	3168.700
1.25 × 100-yr	605.980	3600.800

the conclusion of this analysis, the results in Table E9-5(c) have been determined for the watershed and channel described above.

Figure E9-5(c) illustrates the outflow hydrograph at the DS Junction including the two component hydrographs resulting from the Channel

**Figure E9-5(c)**

Outflow hydrograph at DS Junction, with component hydrographs for preexisting conditions HEC-HMS model. Peak outflow used in HEC-RAS = 2880.7 cfs.

**Figure E9-5(d)**

100-year water surface elevation for preexisting conditions ($RS = 0$).

flow and the Downstream subbasin runoff. Since the Upstream and Downstream subbasins have the same runoff characteristics, the impact of the channel can be seen by comparing the Downstream subbasin runoff with the Channel outflow. The channel causes a shift of about 2.5 hr and a reduction in peak flow of about 43 cfs due to the timing lag and attenuation of the flood wave as it propagates down the channel. Figure E9-5(d) is a cross-section plot of RS 0 showing the 100-yr WSEL.

Stage 2: Existing Conditions Demonstrating Effects of Urbanization

For Stage 2, demonstrating the effects of urbanization, the only changes to the watershed are that development has caused a reduction in the time of concentration (T_c) and storage coefficient (R) associated with the two HEC-HMS subbasins. Each of these values is cut in half for the example, to 0.5 hr and 2.5 hr, respectively. Reducing T_c and R causes the peak flows to rise and to respond more quickly. The new HEC-HMS peak flows cause the water surface elevation in HEC-RAS to rise, changing the values (but not the general shape) of the storage–discharge relationship. After running the SvsQ relationship iterations again as described in Stage 1 above, but now with the reduced T_c and R values, the HEC-HMS and HEC-RAS model outputs are shown in Figs. E9-5(e) through E9-5(g) and Table 9-5(d).

A note about the nomenclature used in several of these figures: HEC-HMS and HEC-RAS have several built-in graphs that are used below, and each can show an “Observed Flow” on the graph—a very convenient feature as utilized herein. In each of the graphs, “Observed Flow” refers to the flow from a previous model run. In this example, each figure caption indicates which previous model run is being shown as an “Observed Flow.”

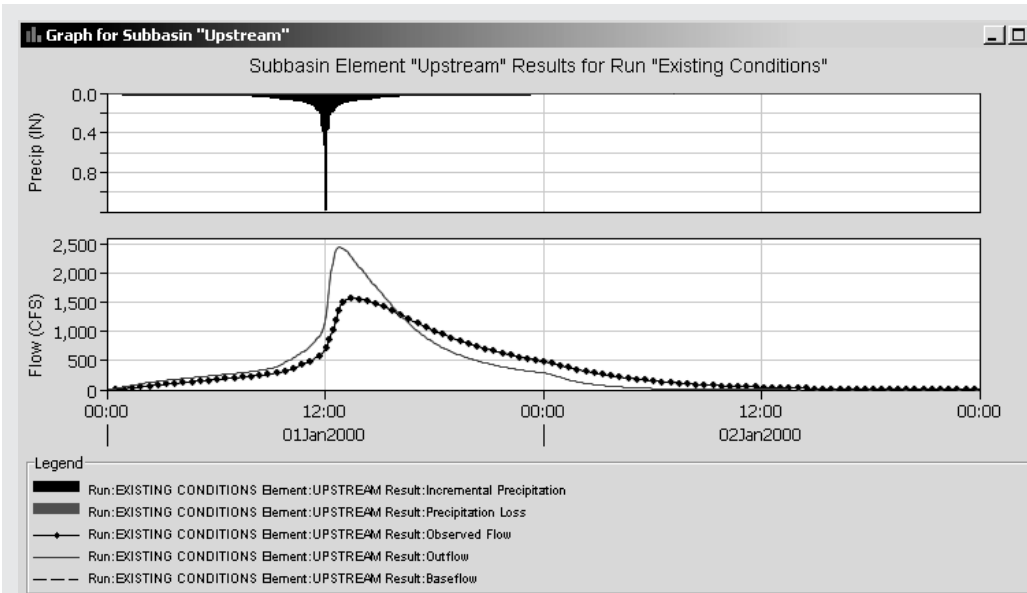


Figure E9-5(e)

Increase in subbasin peak flow due to urbanization. Preexisting conditions shown as "Observed Flow." Increase in subbasin flow approximately 1000 cfs.

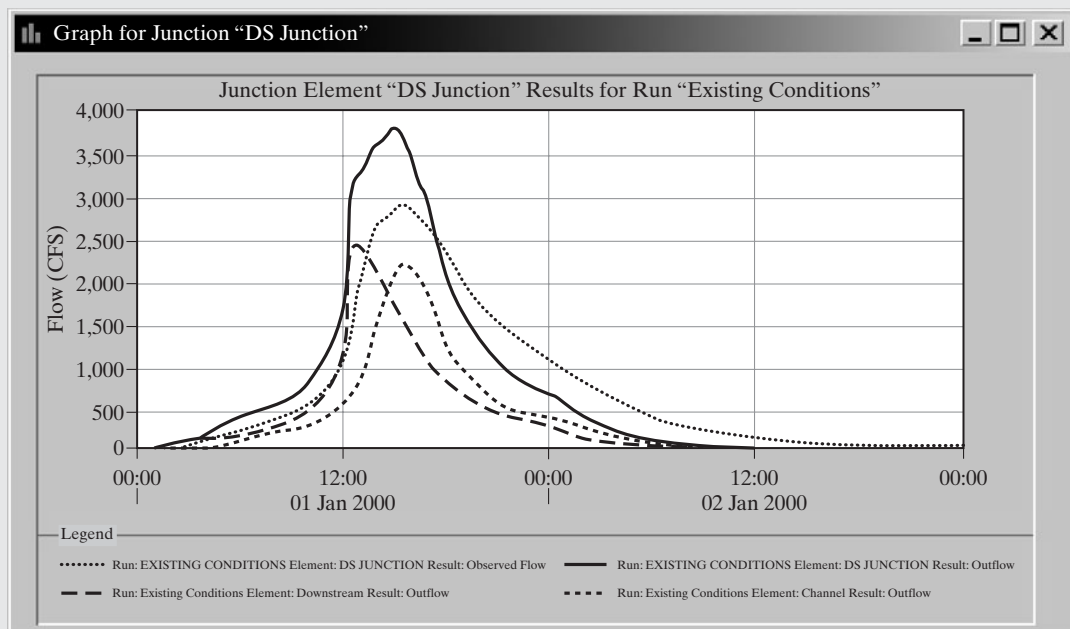
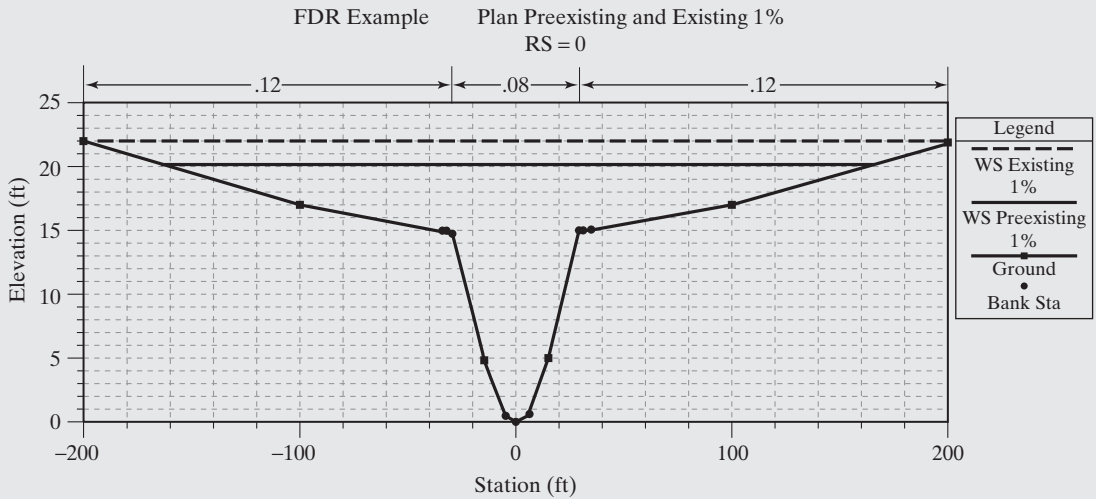


Figure E9-5(f)

Increase in total peak flow at DS Junction due to urbanization. Preexisting conditions shown as "Observed Flow." Note that the total increase in peak flow is about 1000 cfs, which is the same as the increase for the subbasin flow. This is due to the timing and attenuation of the flows in the channel. The peak flow under Existing (urbanized) Conditions is 3801 cfs.

**Figure E9-5(g)**

Increase in WSEL due to urbanization.

Table E9-5(d). Storage–Discharge Relationship for Channel in Existing-Conditions HEC-HMS Model

HEC-RAS Profile	Storage (ac-ft)	Discharge (cfs)
N/A	0.000	0.000
0.01 × 100-yr	13.680	38.010
0.05 × 100-yr	43.620	190.050
0.1 × 100-yr	71.790	380.100
0.2 × 100-yr	119.300	760.200
0.4 × 100-yr	221.610	1520.400
0.6 × 100-yr	367.620	2280.600
0.8 × 100-yr	507.050	3040.800
0.9 × 100-yr	574.630	3420.900
1.0 × 100-yr	640.370	3801.000
1.1 × 100-yr	700.500	4181.100
1.25 × 100-yr	781.050	4751.250

Stage 3: Proposed Conditions with Channel Improvements

Next, channel modifications are performed to develop a channel geometry that can carry all of the flow from the existing system within banks. The intention to perform channel improvements to invoke flood damage reduction (FDR) in the overbank regions, as shown in Figure 9–16. Channel improvements (modifications) can be computed by HEC-RAS using the Channel Modification tool to calculate the new bank stations and elevations, which is a very convenient feature. In essence, we are leaving the

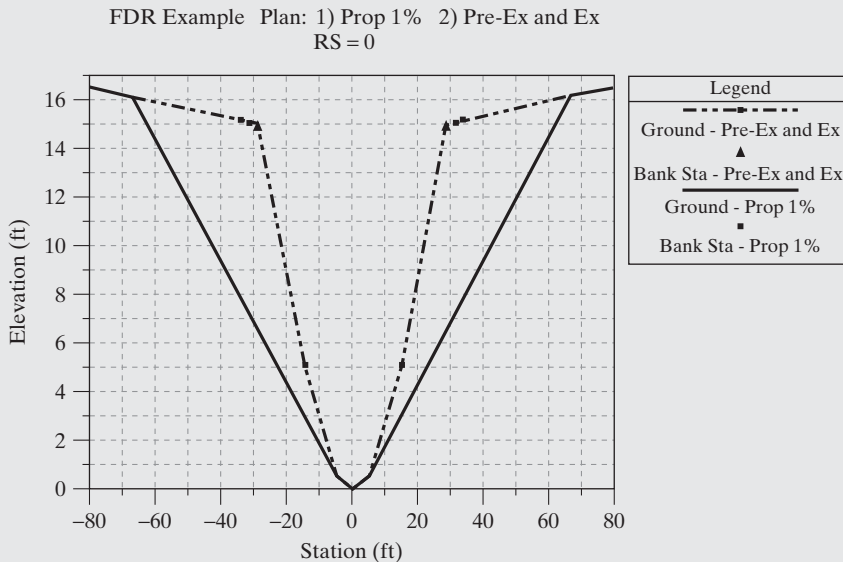
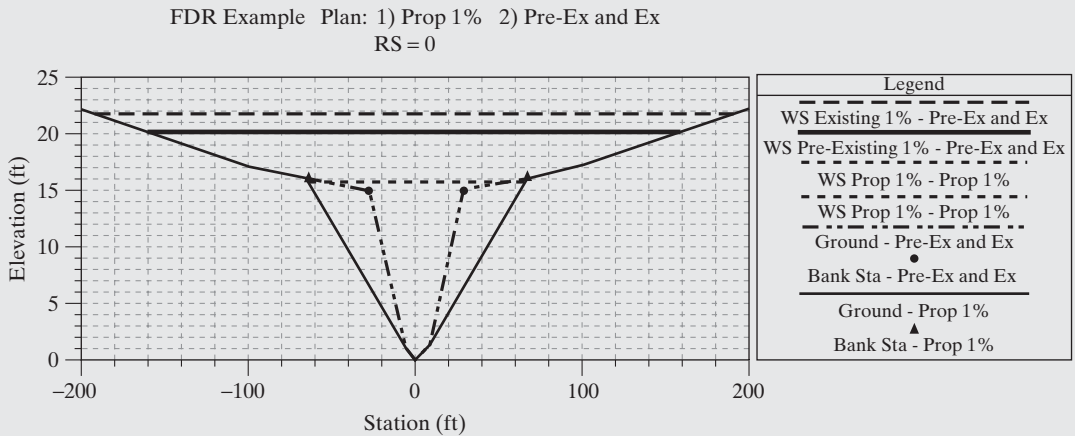


Figure E9-5(h)

Channel modifications for Proposed conditions. Note that the bottom three points of the Proposed section match the Existing section.

bottom three points of the existing cross section the same in terms of station and elevation and then we are cutting back the side slopes at 4:1 to meet the existing overbank regions. Using the HEC-RAS Channel Modification tool in the geometry editor, the following modification parameters to the Existing-conditions channel cross sections [see Fig. E9-5h] can be invoked: Bottom width = 10 ft; left and right slopes = 4:1; Manning's roughness = 0.040; the new cut channel invert elevation (not to be confused with the channel flow-line invert elevation itself—in other words, the points where the 4:1 cut side slope begins) = 0.5 ft up from the channel flow-line elevation. The channel cut should be projected from the lower RS at the same slope as the existing conditions (slope = 0.00075 ft/ft). Note that the proposed channel's bottom section is the same as the existing channel, as shown in Figure E9-5(h), but the channel banks have moved out due to the 4:1 side slope channel cuts. The HEC-RAS User's Manual gives a good description of the Channel Modification tool and its use. The roughness along the bottom of the channel in the new geometry file should be changed to the new roughness value of 0.040. The overbank roughness remains unchanged. Once the channel improvements are made in HEC-RAS, SvsQ iterations are again performed on the new channel improvement geometry, resulting in a peak flow of 4267.5 cfs.

**Figure E9-5(i)**

Channel modifications and WSELs for Preexisting, Existing, and Proposed conditions.

Figure E9-5(i) shows the existing and proposed channel banks, as well as the preexisting, existing and proposed WSELs for the analysis. Notice the proposed WSEL (Prop 1%) is now below the banks of the channel. Table E9-5(e) lists the storage–discharge relationship produced after the SvsQ iterations are performed again, and Table E9-5(f) is a summary table of the flow and WSEL results for the Preexisting-, Existing-, and Proposed-conditions models.

Looking at RS 0, once the channel improvement modifications are in place, the water surface elevation is reduced to 15.54 ft, from an Existing-conditions level of 21.76 ft. The new channel banks at this RS are at an elevation of 16.06 ft, so the new project has 0.52 ft of freeboard above the 100-yr

Table E9-5(e). Storage–Discharge Relationship for Channel in Proposed-Conditions HEC-HMS Model.

HEC-RAS Profile	Storage (ac-ft)	Discharge (cfs)
N/A	0.000	0.000
0.01 × 100-yr	9.940	42.675
0.05 × 100-yr	32.730	213.380
0.1 × 100-yr	54.900	426.750
0.2 × 100-yr	92.200	853.510
0.4 × 100-yr	154.920	1707.000
0.6 × 100-yr	209.930	2560.500
0.8 × 100-yr	260.460	3414.000
0.9 × 100-yr	284.490	3840.800
1.0 × 100-yr	307.790	4267.500
1.1 × 100-yr	330.130	4694.300
1.25 × 100-yr	361.640	5334.400

Table E9-5(f) Summary Flow and WSEL Data for Preexisting-, Existing-, and Proposed-Conditions Models

River Station	Profile	Q Total (cfs)	WSEL (ft)
2.4	Preexisting 1%	2881	29.72
2.4	Existing 1%	3801	31.27
2.4	Proposed 1%	4267	25.05
0	Preexisting 1%	2881	20.21
0	Existing 1%	3801	21.76
0	Proposed 1%	4267	15.54

peak WSEL. These channel modifications, however, cause the peak outflow from this reach of channel to increase from 3801 cfs to 4267 cfs, about 12%. At the same time, the storage in the overbanks is reduced from about 440 ac-ft to 0 ac-ft, as there is no flow or storage in the proposed overbank regions, while the volume below the new channel banks is increased from about 200 ac-ft to about 307 ac-ft. This increase in flow rate and the corresponding total reduction in storage of about 333 ac-ft will cause a significant impact downstream of this section of channel, as previously described in this chapter. In an actual detailed project situation, the downstream channel (or receiving stream) would be modeled as part of this study to determine whether there is no adverse impact. For this example, we will assume that if the peak flow rate can be reduced back to the Existing-conditions rate of 3801 cfs from 4267 cfs, the impacts of the channel modifications will have been fully mitigated. The above steady-state analysis can be expanded to include unsteady state as shown below, but this presents a more complex solution.

Stage 4: Mitigated Proposed Conditions with an Off-Line Detention Basin

The final stage of the example is the introduction of an off-line detention basin, similar to the one illustrated in Figure 9-17, to mitigate the impacts of the channel improvement modifications introduced in Stage 3. Detention basins can be modeled by several methods, including steady-state diversion and detention in HEC-HMS. In this analysis, we use a more sophisticated unsteady modeling application in HEC-RAS to provide a view of advanced design techniques. A main benefit of using unsteady HEC-RAS at this point is that once the unsteady flow data is entered into the model, a large number of trials can be easily run to examine the behavior of a detention basin in various configurations and sizes. All of the channel and detention basin routing is performed in HEC-RAS. To ensure the best comparison between conditions with and without the detention basin, Existing conditions are also modeled in HEC-RAS unsteady-state mode.

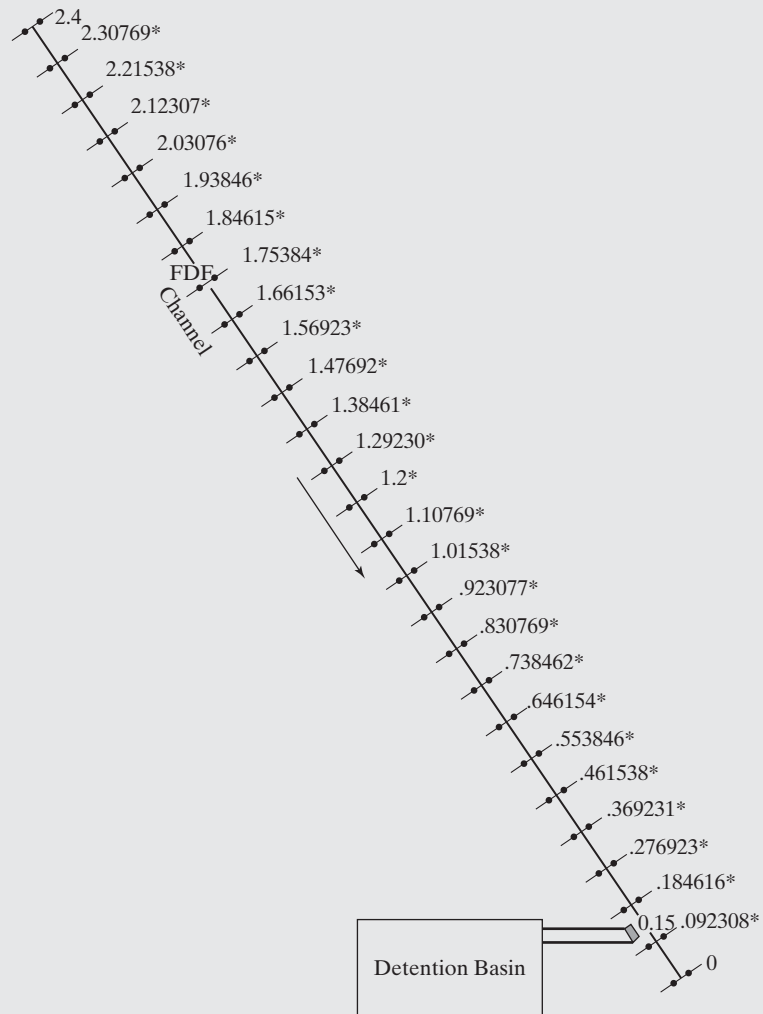
The detention basin is to be a peak “cut-off” or “shaving” basin, as previously described in this section, designed to reduce the peak outflow

in a channel by removing water from the top of the hydrograph. An off-line detention basin, like that shown in Figure 9–17, with a side-flow weir will perform this task most efficiently, as the basin will not begin to fill until the hydrograph is already rising, and will, if designed properly, reduce the peak flow rate downstream of the basin.

For the unsteady HEC-RAS model, several changes to the Existing-conditions geometry are necessary. None of them change the results when run with the steady-flow profiles. First, some cross sections need to be interpolated, with a maximum spacing of 500 ft, to improve model stability, allow for assignments of flows along the channel, and allow for the placement of the detention basin. HEC-RAS cross-section interpolation assumes that there are linear changes between the end cross sections and allows the user to determine which points along the channel are ‘equivalent.’ For this example, since the original cross sections are the same, all of the interpolated cross sections will have exactly the same shape, but will be shifted vertically, and the distances between the cross sections will be adjusted. When interpolating cross sections, the user enters a maximum spacing, and HEC-RAS calculates the largest spacing for an equally spaced set of interpolated cross sections just closer together than that maximum. Reach stations for the new cross sections are calculated from the upstream and downstream reach stations, and interpolated cross sections are marked with an asterisk (*) at the end of their names. There must be two cross sections between most boundary conditions, such as bridges, weirs, lateral structures, and the upstream and downstream ends of river reaches, so more cross sections than actually necessary were used to allow the detention basin to be placed closer to the end of the channel. The results of the cross-section interpolation and the addition of the off-line detention basin are illustrated in Figure E9–5(j).

All of the routing in the unsteady model will be performed by unsteady HEC-RAS, thus eliminating the SvsQ iterations necessary with the steady-flow models. The outflows from the Upstream and Downstream subbasins will be applied at the appropriate locations along the channel. Since the outflow from the subbasins for Existing and Proposed is the same for developed conditions, flows from either one can be used for the unsteady analyses. The Upstream subbasin’s flow should be applied at the upstream end as a flow hydrograph, and the Downstream subbasin’s flow should be applied as a uniform lateral inflow hydrograph between RS 2.21538* and RS 0.184616*. The downstream boundary condition will be normal depth at a slope of 0.00075 ft/ft. When these changes are made, the unsteady Existing-conditions model can be run. The resulting model has a downstream WSEL of 21.60 ft, which is 0.16 ft lower than the downstream WSEL in the steady-state model. The unsteady model’s peak flow rate is 3704 cfs, which is about 100 cfs lower than the flow rate from the steady-state model. It is typical for unsteady models to differ slightly from their steady counterparts due to changes in routing and timing of the hydrographs. These

Figure E9-5(j)
HEC-RAS
schematic
showing the
detention basin
and the weir
connecting it to
the channel.
HEC-RAS requires
two cross sections
between a
structure and the
model boundary
(RS 0).



slight differences are deemed suitable for the utilization of HEC-RAS unsteady in development of the detention basin for this project example. Figure E9-5(k) shows the downstream hydrographs from the two Existing-conditions models (steady vs. unsteady). The change in timing is clearly shown, with the unsteady model having a slower rise and lower peak, but the same time to peak as the steady model. These differences are why the Existing-conditions model was converted to unsteady mode such that we can make appropriate comparisons, using the unsteady model data, between the Existing- and Mitigated Proposed-conditions models.

Next, the same cross-section interpolations are made to the Proposed-conditions geometry file with the channel modifications, and the same

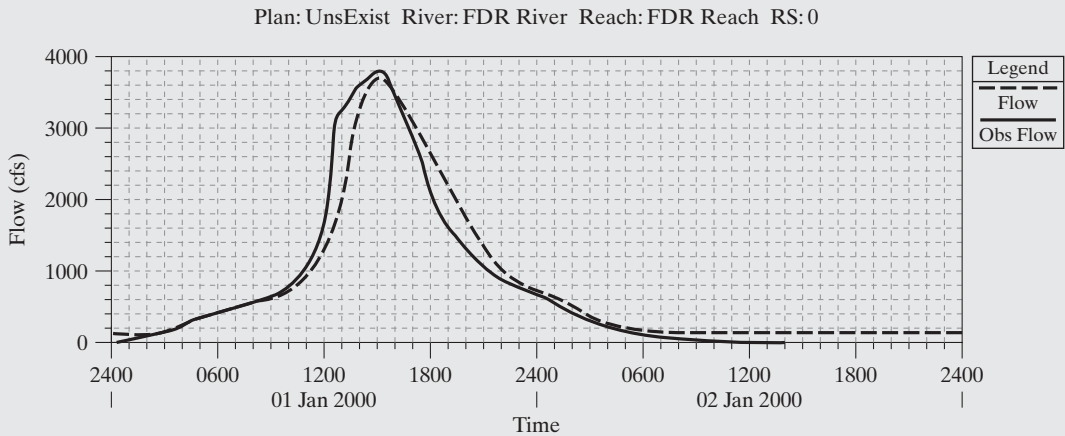


Figure E9-5(k)

Downstream hydrograph from steady and unsteady models. Steady model is shown as "Obs Flow."

boundary conditions are applied. At this stage, a model run of the Proposed conditions without any mitigation can be run. The results of this run are shown later along with the results of the run that includes the detention basin (the Mitigated Proposed conditions).

Finally, a storage area and lateral structure can be added in HEC-RAS. After several model runs varying the detention volume, the weir elevation, and the weir width, a 120 ac-ft basin with a 130-ft weir was designed. This result can be determined by many means, but a trial-error process is easy to apply and very suitable. The detention basin storage area's elevation vs. volume data are shown in Table E9-5(g), and the lateral structure's weir is defined in Table E9-5(h). The lateral structure will be at RS 0.15 and will flow into the storage area. The weir should be 182 ft from the upstream cross section, with a weir coefficient of 2. To allow the detention basin to drain at the end of the storm, a 36-in. corrugated metal pipe (CMP) culvert will be installed at the downstream end of the weir (120 ft from the upstream end). The 100-ft length of CMP has a roughness

Table E9-5(g). Detention Basin Storage Area Elevation vs. Volume Data for Proposed-Conditions Unsteady HEC-RAS Model

Elevation (ft)	Volume (ac-ft)
0.61	0
1	1
5	50
15	120
17	122

Table E9-5(h). Detention Basin Lateral Structure (Weir) Station vs. Elevation
Data for Proposed-Conditions Unsteady HEC-RAS Model

Station (ft)	Elevation (ft)
0	17
16	13
146	13
162	17

of 0.024, an entrance loss coefficient of 0.5, an upstream invert of 0.61 ft, which is also the bottom or wet basin elevation of the detention basin, and a downstream invert of 0.51 ft, which is approximately the channel invert at the outfall location. Finally, the culvert should have ‘No Positive Flow Flaps’ applied, which will keep water from entering the reservoir through the culvert. Without the flap gate, the basin will begin to fill during the rising limb of the storm, thus decreasing its effectiveness.

Figure E9-5(l) shows the resulting hydrographs at the downstream end of the channel. The “UnsNoDet” model results show the impact of channelization without including detention (i.e., the Proposed conditions with the FDR channel improvements in place). The “UnsDet” model results show the Mitigated Proposed conditions with the detention basin in place. The time to peak of the hydrograph shortens by 75 minutes, and the peak flow increases by about 700 cfs when comparing the Proposed-conditions (“UnsNoDet”) hydrograph with the Existing-conditions (“UnsExist”) hydrograph. Introducing the detention basin reduces the peak flow rate

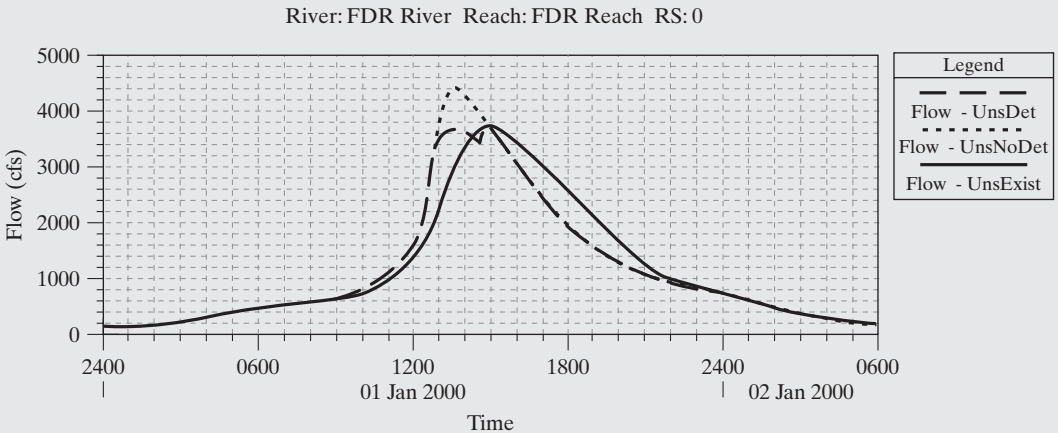


Figure E9-5(l)
Downstream hydrographs comparing Existing (“UnsExist”), Proposed with no detention (“UnsNoDet”), and Mitigated Proposed with detention (“UnsDet”).

of the Proposed- to the Existing-conditions level, though the time to peak is still much earlier. By reducing the peak flow rate [i.e., the Mitigated Proposed peak flow rate—"UnsDet" in Fig. E9-5(l)] to that of the Existing conditions, the project example is deemed fully mitigated of adverse impacts downstream.

Changing any of the three primary weir/basin parameters (length of the weir, elevation of the weir, and volume of the detention basin) will change the general behavior of the basin as a whole. Increasing the length of the weir will cause it to fill faster, lowering the weir will cause it to start filling sooner, and changing the volume of the basin will change the time when the basin is full.

In this example, the basin fills up before the flow falls back to the Existing-conditions peak flow, allowing a second 'peak' in the final hydrograph [see Fig. E9-5(l)]. Reducing the weir length will increase the height of the first peak, and the water surface will recede before filling the basin completely. Increasing it will reduce the first peak, but the basin will fill before the upstream hydrograph falls below the Existing flow rate, causing that higher flow rate to be passed downstream. Experimentation with variations in weir/basin geometry, as well as the elimination of the outlet conduit flap gate, is encouraged at this point to develop insights into how these items affect the operation and effectiveness of the basin.

Figure E9-5(m) shows the maximum water surface profiles along the channel for the same three models. Introducing the detention basin

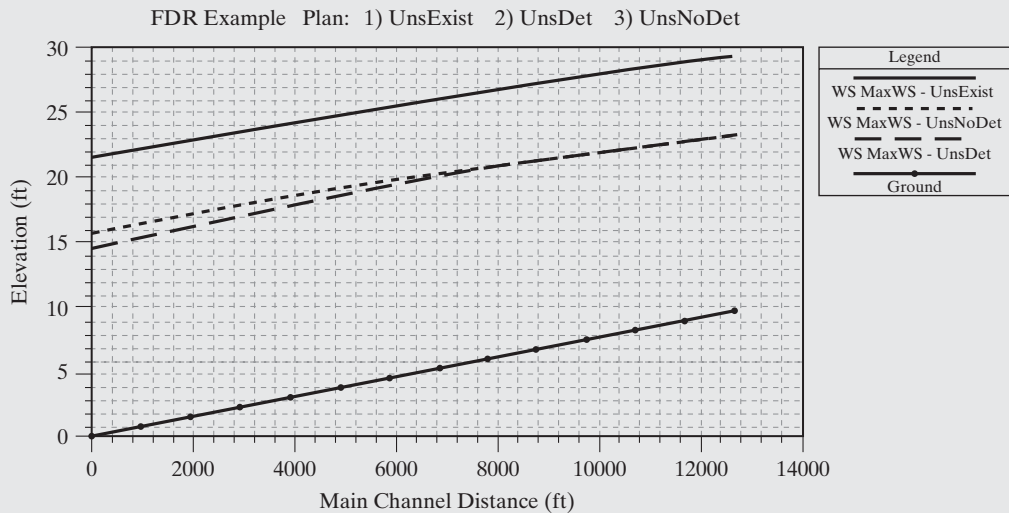


Figure E9-5(m)

Channel profiles of unsteady Existing ("UnsExist"), Proposed without detention ("UnsNoDet"), and Mitigated Proposed with detention ("UnsDet") models. The "Ground" profile illustrates the channel slope.

Table E9–5(i). Summary Flow and WSEL Data for Unsteady Preexisting, Existing, Proposed, and Mitigated Proposed Conditions Models

River Station	Profile	Q Total (cfs)	WSEL (ft)
2.4	Preexisting	1471	27.21
2.4	Existing	2186	29.07
2.4	Proposed	2322	23.00
2.4	Mitigated	2339	22.98
.184616*	Preexisting	2728	20.64
.184616*	Existing	3705	22.33
.184616*	Proposed	4465	16.54
.184616*	Mitigated	3645	15.33
0	Preexisting	2728	19.91
0	Existing	3704	21.60
0	Proposed	4465	15.82
0	Mitigated	3697	14.68

reduces the flow rate at the downstream end, which reduces the WSEL at that stage as well. This example, however, does not account for the channel conditions downstream of the project. Understanding the full impact of a given channel modification project would require modeling of the full downstream reach as well unless the outfall is into a large reservoir or bay for which the downstream boundary condition is essentially an infinitely large hydraulic sink. Table E9–5(i) lists the final summary results of all four stages of this example as run with unsteady HEC-RAS (Preexisting, Existing, Proposed, and Mitigated Proposed) for review. Note that the “Mitigated” flow at RS 0 is slightly less than the “Existing” flow, indicating the mitigated conditions.

SUMMARY Chapter 9 presents design applications in hydrology with detailed explanations, giving examples of the design procedures commonly used in urban, rural, and floodplain hydrology applications. The design development of drainage collection systems using the rational method is presented along with the application of synthetic design storms. The design explanation continues with the presentation of the concepts of overland flows combined with underground conveyance systems. This is a very complex subject, as it involves the dynamic nature of urban storm water management. Example 9–1 provides a detailed example of the design of an urban storm sewer reconstruction project, and the HGL is developed and plotted in Figure 9–7 to illustrate the completed design; however, a more detailed design including the design of street inlets would usually be performed. Minor losses for storm sewers as well as the behavior of inlets are discussed at the conclusion of Section 9.2.

In Section 9.3 the theory of culvert behavior is presented, and Example 9–2 is a detailed example using hand calculation methods for a culvert under outlet control.

There is much more information on culvert analysis and design, and other reference sources are identified to develop a better understanding of culverts. More complex culvert design and analysis is presented, and an example of a culvert under inlet control modeled using HEC-RAS is included. Considering the impacts of urbanization, detention basin design is included in Section 9.4 along with a detailed example. The procedures in this example can be input into a spreadsheet as a convenient tool for detention basin sizing. In Section 9.5 floodplain management design issues are presented with a complete flood damage reduction project example. It is required that the user of this final example be rather familiar with HEC-HMS and RAS, as the various steps use this software extensively. The unsteady HEC-RAS modeling techniques presented in Example 9–5 are very much real applications, and they provide an excellent opportunity for the user to understand the relationships of storage and routing in floodplain management design and analysis.

PROBLEMS

- 9.1.** Confirm the data for a 10-yr storm from Equation (9–2) and Table 9–2 for 10-min, 15-min, 30-min, and 60-min durations. Compare to Figure 9–4.
- 9.2.** Construct the 100-yr, 24-hr synthetic design hyetograph using Equation (9–2) and 1-hr time intervals. Center the maximum intensity (the 1-hr intensity) at hr 12; the next highest intensity (determined by the depth from 2 hr – 1 hr over the 1-hr interval—hence, intensity) at hr 11; the next-highest intensity (depth from 3 hr – 2 hr) at hr 13; the next-highest intensity (depth from 4 hr – 3 hr) at hr 10; and back and forth similarly. Check the total volume under the hyetograph and see that it matches the total rainfall depth.
- 9.3.** Determine the runoff Q using Equation (9–1) for a project with $A = 30$ ac, $C = 0.65$, $T_c = 12$ min, and an i -value developed from Equation (9–2) for a 5-yr design storm.
- 9.4.** Determine the smallest required concrete round pipe and rectangular box sewers for $Q = 125, 175, 225$, and 275 cfs, $S = 0.005$ ft/ft, and $n = 0.013$ using Equation (9–3). Tabulate results and compare capacities and areas of the round to box sewers. Use standard increment sizes of 0.5 ft for round pipe and 1.0 ft for box sewers.
- 9.5.** Derive Equation (9–4) from Equation (9–3).
- 9.6.** For a $Q = 100$ cfs, calculate the $h_f =$ for a 3.5-ft-diameter, 500-ft-long, round pipe sewer with $n = 0.012$ flowing full.
- 9.7.** Using the same flow and the same 500-ft-long round pipe sewer as in Problem 9.6, determine and plot the HGL and EGL for the sewer reach, given the following additional information: U/S pipe invert elevation = 100.00 ft; D/S pipe invert elevation = 96.00 ft; and the tailwater elevation for the sewer (i.e., the outlet water surface elevation) = 106.00 ft.
- 9.8.** Referring to Table E9–1(a), double the flows in Column 11 for each sewer reach and determine the minimum commercially available concrete box sizes required (available in 1.0-ft increments) for each sewer reach using a minimum box span and rise of 2.0 ft. Keep all elevation, length, slope, and roughness data the same as presented in the table. After sizing the box sewers, recompute the

HGL for the entire system. Evaluate any problems with maintaining a minimum actual velocity of 3 fps.

- 9.9. Calculate the manhole losses, h_f , of 90-, 60-, 45-, 30-, and 15-degree manhole junctions with no special deflector for a 4.5-ft-diameter round pipe sewer flowing full with $Q = 150$ cfs. Tabulate and compare the results.
- 9.10. Given the inlet in Figure 9-9 with $h = 0.5$ ft and $W = 1.5$ ft, determine what minimum length, L , of inlet (in whole-foot increments) is required for $Q = 8$ cfs, such that the depth of flow at the curb does not exceed 0.5 ft.
- 9.11. Given the inlet in Figure 9-9 with $h = 0.5$ ft, $W = 1.5$ ft, $L = 4.0$ ft, and a gutter depression depth = 0.33 ft, determine what is the Q into the inlet if the depth of storm water at the curb is 1.0 ft (i.e., there is 6 inches of water depth above the standard curb height of 6 inches for a total depth at curb of 1.0 ft).
- 9.12. Repeat Example 9-2 using 3-ft-diameter round corrugated metal pipe culverts with an $n = 0.024$. Determine the minimum number of culvert barrels needed to convey the flow without overtopping the roadway.
- 9.13. Repeat Example 9-3 using HEC-RAS but use a 100-yr storm at this location. Recalculate Q for a 100-yr storm using the same C , A , and T_c information provided in Example 9-2. Evaluate the behavior of the 7×3 box culvert and then examine the behavior of a 6×3 box culvert at this location. Document the differences in culvert performance, including weir flow depths for comparative purposes. Determine the minimum span (width) of box culvert needed to prevent overtopping of the roadway if the box culvert rise (height) allowed is increased to 4 ft.
- 9.14. Repeat Example 9-4 but with the following changes to the data:
 - (a) SCS TR-20 methodology data
 - (i) Existing (preproject) $T_c = 1.3$ hr; changed from 1.0 hr
 - (ii) Proposed (postproject) $T_c = 0.4$ hr; changed from 0.45 hr
 - (b) Figure E9-4(a) data
 - (i) Emergency spillway and basin design elevation = 229.50 ft; changed from 231.00 ft
 - (ii) Maximum basin depth = 14.0 ft ($229.50 - 215.50$); changed from 15.5 ft

All other data and information remain unchanged.

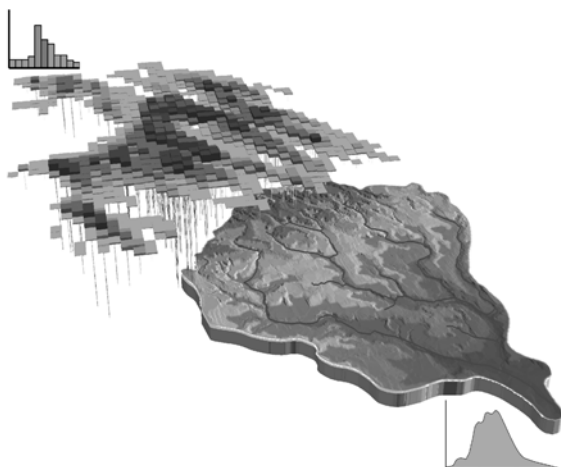
- 9.15. Repeat Example 9-5 using the same data except that the two subbasins (Upstream and Downstream) are 2.85 mi^2 each now (changed from 2.0 mi^2 each). All other data remain unchanged. This will necessitate extending the existing overbank regions of the Preexisting- and Existing-conditions cross sections along the same slopes to properly analyze the behavior of the channel in steps 1 and 2. In step 3, increase the bottom width of the Proposed cross section beyond 10 ft, keeping the same 4:1 side cross slopes, until the flow is contained within the channel banks. Adjust the weir and detention basin geometry as needed in step 4 to fully mitigate the impacts, as described in the example.

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Chapter 10

GIS Applications in Hydrology



GIS map of NEXRAD rainfall over a watershed



LiDAR digital elevation data shown here with hillshading reveals topographic information used for hydrologic modeling (Photo courtesy Vieux and Associates)

10.1 INTRODUCTION TO GIS

A geographical information system (GIS) includes: (1) attribute or feature information in a database format, (2) locational information for these attributes or features, and (3) analysis functions for deriving new information on spatial relationships. GIS has a wide application in a variety of civil and environmental applications, including demography, meteorology, transportation, urban planning, and hydrology. In the past three decades, hydrologic applications have been revolutionized by GIS, making it an indispensable technology for digital terrain and hydrologic analysis of watersheds. The role and usage of GIS technology, software, data formats, and sources of geospatial data are essential to hydrologic analysis as covered in this chapter.

Common to all definitions of a GIS is the concept of linking data with a geographical location, which is the definition of **spatial data**. Attributes within the context of GIS refer to the characteristic(s) mapped at a specific location. For example, attributes could be the population and demographics in a city block, or the slope of the land surface, or the land use for a given area. Successful use and adoption of GIS technology for any purpose depends on factors that extend beyond the data and the computer on which

they reside. Effective use of a GIS system requires resources devoted to the computer hardware, GIS software, and user training in data characteristics and spatial analysis techniques. This chapter describes the background and use of GIS in hydrologic analysis and relates common GIS terminology and functions to specific hydrologic applications.

A GIS can record observations or measurements, which can be thought of as **features**, **activities**, or **events**. A **feature** is a term used in digital cartography to refer to an item or piece of information placed on a map. Point features such as rain gages or a benchmark have location with no dimension (0-D), while line features such as a watershed boundary or river have several locations strung together to form an ordered sequence of points that are one-dimensional in space (1-D). Area features, such as the area representing a watershed or floodplain boundary, consist of lines that form a closed polygon, i.e., a two-dimensional feature (2-D). Besides physical features, human **activities** are often described with geographical patterns and spatial distributions. Population maps, census maps, and urban infrastructure maps, e.g., sewers and water distribution networks, are examples of human activities that can be mapped within a GIS. The temporal dimension of GIS data requires a definition of *when* something occurred, termed an event. Mitchell (1999) defines an **event** as something that occurs at a point in time and can be mapped over time. For example, a GIS database containing a time series of rainfall values at a point or over a watershed can be used to identify the accumulation of rainfall depth, duration and intensity in the selected time interval, summation during storm event, or long-term amounts such as annual precipitation. A GIS database containing spatial location, attributes, and temporal records supports queries and analysis of the spatial and temporal dimension of hydrologic quantities such as rainfall.

Ian McHarg (McHarg, 1995) popularized the use of GIS in landscape design in the 1970s. Its employment in census and population demographics and geography or urban planning disciplines gained recognition before its application to hydrology evolved. Like the adoption of electronic spreadsheet technology, first developed for accountants, and later used extensively in other scientific and engineering analysis, the application of GIS began in geography and has rapidly been adopted by other scientific disciplines. GIS is widespread and established in hydrologic applications, for which its acceptance and growth were accelerated by the availability of:

- Worldwide geospatial data, such as soils, topography, and land use/cover
- Inexpensive computer hardware capable of performing complex and detailed computations
- Training and education targeted to the engineering community
- Generally available GIS software with extensions tailored for hydrologic analysis

Active involvement in hydrology by the GIS community is represented today by user groups devoted to GIS technology applications to hydrology, natural resources management, or urban planning. National hydrographic and elevation datasets are available from the U.S. Geological Survey (USGS), digital soils data from the Natural Resources Conservation Service (NRCS), and abundant other spatial data formatted for GIS use are available from other federal and state agencies. Universities introduced GIS training into their engineering curricula to bridge the gap between technology and education, and technology centers offer training for technicians that can perform routine GIS functions.

In hydrology, GIS is used for watershed delineation, runoff estimation, hydrologic and hydraulic modeling, and floodplain mapping. Digital representation of topography, soils, land use/cover, and precipitation may be accomplished using GIS data and methodology. These applications are enhanced through the use of GIS, because hydrology is inherently spatial in nature.

Data types can differ, depending on the scale at which survey data and public GIS data are collected and compiled. The resolution and spatial detail found in generally available geospatial data may be appropriate for hydrologic applications when dealing with large basins. However, in smaller watersheds, the scale, spatial detail, and resolution may not be sufficient. Considerations of map scale and resolution are presented in Vieux and Thompson (2004) along with the standards for scale and resolution of GIS, remotely sensed imagery, and surveyed data collected for a variety of applications including hydrology.

The sections that follow provide an introduction to GIS concepts, data structures, geographic projections, and spatial analysis techniques applicable to hydrology. Further details may be found in the excellent textbooks and references mentioned and through online sources (see Appendix E).

10.2 GENERAL GIS CONCEPTS

Data used in GIS generally have a structure and coordinate system that is formed from **vector segments**, e.g., lines, points, or polygons; or is composed of a regular array of rectangular grid cells referred to as a **raster**. For example, digital representation of land surface elevations may be in the form of contour lines (vectors) or of a grid of elevations at regularly spaced intervals (raster format). The coordinate system used to locate vector or raster data is referenced to the earth surface so that the distance between features and spatial relationships between them can be derived. Data structures, map scale and spatial detail, datum issues, and coordinate systems are general concepts needed for understanding GIS usage and application to hydrology. A basic understanding of these issues is necessary to effectively utilize GIS systems for analysis of hydrologic information. Each of these topics is discussed below in further detail.

Raster and Vector Data

GIS data are usually formatted in a **vector** or **raster** structure. A vector dataset maps a geographic feature using lines drawn between points or coordinate pairs. An advantage of vector formats is that a database containing multiple attributes may be associated with any given point, line, or polygon, whereas in the raster format only a single attribute is usually associated with a grid cell. Vector data are used to show rain gages as points, streams as lines, and the watershed boundary as a polygon in Figure 10–1 (left). On the right, a raster of grid cells represents rainfall in relation to the watershed and other features in the map. Vector format supports the ability to store topology of the data, which defines the order and relationship among vector data. For example, a vector data structure containing topologic information could be used to represent a stream network within a watershed. A GIS system containing the data could be used to query and search for all downstream segments affected by a spill or discharge of pollutants within the stream network. Raster data are used to describe spatially continuous information and are usually sampled on a regularly spaced grid, as seen in Figure 10–1 (right). Examples of raster datasets are digital elevation models (DEMs), gridded radar rainfall data, or land uses from satellite imagery. The vertical resolution of the data and the reliability of the resulting delineated stream network for watersheds are described by Gyasi-Agyei et al. (2006).

In many hydrologic applications of GIS, a combination of both vector and raster data is needed, depending on the specific application or data source. For example, mathematical and/or feature manipulation of two datasets to create a new dataset could be used to derive rainfall estimates over a watershed. The intersection of NEXRAD radar data (raster) and subwatershed delineations (vector) can be performed to calculate rainfall depth over each subwatershed. This concept is illustrated in Figure 10–1 (right), where gridded radar rainfall is overlaid on the watershed area of

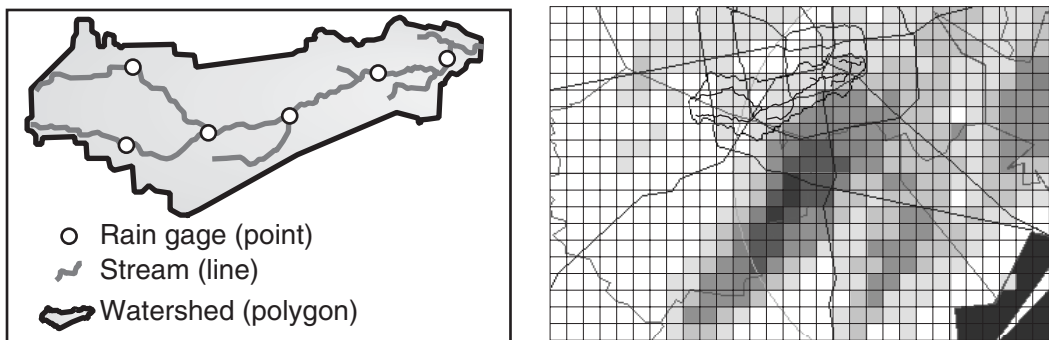


Figure 10–1

Vector GIS features can be points, lines, or polygons (left panel), while raster GIS data represent a gridded array of continuous data (right panel).

Brays Bayou in Houston, Texas. Analysis of GIS data can involve raster–raster, vector–vector, or raster–vector manipulations. Spatial analysis techniques such as intersection, union, mathematical computation, buffer generation, dissolving boundaries, tabular data analyses, network analyses, and digital terrain modeling make GIS indispensable in hydrology as in many other disciplines. Such spatial operations create new knowledge about relationships between data layers.

Map Scale and Spatial Detail

The scale and resolution at which the data are collected or measured on a map is termed the **native scale** or resolution. Unlike paper maps, map scale is easily altered within a GIS system. Therefore, a distinction should be drawn between the inherent or native scale of the data and the displayed or printed scale. If the spot or point elevations are surveyed in the field on a grid of 100 m, then this is its native resolution. Once contours are interpolated between the points and plotted on a paper map, a native scale of the data is introduced. For example, if a paper USGS quad sheet is digitized, the native scale will be 1:24,000. As a result, performing an analysis of this data at a 1:1000 scale could adversely influence the accuracy of the analysis.

A small-scale map is one in which features appear small. For instance, maps at 1:1,000,000 have the advantage of covering large areas, but show few details. Conversely, large-scale maps have features that appear large and show significantly more detail. A topographic map can be shown at any scale within a GIS, depending on the needs of the application, provided one does not exceed the accuracy supported by the data.

Datum and Spheroid

Geodesy is the branch of applied mathematics concerned with determining the size and shape of the earth and the exact positions of points on its surface. The earth is often treated as a sphere to make mathematical calculations easier, but it is actually a **spheroid**. The earth must be treated as a spheroid to maintain accuracy in maps of large spatial extent, and a number of standard spheroids are commonly used to describe its shape. In North America, these include the Clarke 1866 Spheroid and the GRS80 (Geodetic Reference System of 1980).

A horizontal datum is a reference frame used to measure locations on the surface of the earth. It defines the origin and orientation of the lines of latitude and longitude. A horizontal datum is always related to a specific spheroid. There are two types of datums: earth centered (geocentric) and local. A local datum is aligned so that it closely corresponds to the earth's surface for a particular area, such as the 1927 North American Datum (NAD27). This datum uses the Clarke 1866 Spheroid, which passes through Meades Ranch, Kansas. Each point on the earth's surface can be defined with a latitude–longitude coordinate pair. Latitude and longitude represent

the angles (N-S and E-W) to that point from the center of the earth. Standard latitudes and longitudes used over the globe are then measured from the center of this spheroid, which does not necessarily correspond with the center of mass of the earth. The Clarke 1866 Spheroid and NAD27 fit North America fairly well, but are not accurate for other continents. An earth-centered datum, such as the 1983 North American Datum (NAD83), is based on the GRS80 spheroid centered at the mass center of the earth, and is therefore more accurate for maps of larger extent than North America. The national geodetic survey is responsible for establishing geodetic datums (see www.ngs.noaa.gov).

From a vertical standpoint, land elevation is measured from a theoretical surface called mean sea level, which is the elevation that the ocean would adopt if it uniformly covered the earth. Because mean sea level cannot be defined inland, a geoid is used as a vertical reference. The geoid is a surface of constant gravitational acceleration (g) over the earth. However, there are problems delineating the geoid due to gravitational anomalies, of which maps have been prepared and are constantly being improved. For North America, the standard vertical datums are the National Geodetic Vertical Datum of 1929 (NGVD29) and the North American Vertical Datum of 1988 (NAVD88). Care should be exercised when using geospatial data compiled in different datums. Geospatial data in one datum must be adjusted through reprojection to be consistent with features mapped in another datum. The datum used to compile data and maps in a GIS should be documented to avoid problems associated with inconsistent vertical and horizontal datums.

Geographic and Cartesian Coordinate Systems

A geographic coordinate system is based on latitude and longitude, as defined above. A transformation or **projection** from a curved geographic coordinate system to a Cartesian coordinate system is necessary to view all or part of the earth as a flat map. Because it is impossible to represent a three-dimensional figure such as the earth as a two-dimensional map with minimal distortion, various projections have been developed to preserve one or more of the following properties: shape, area, distance, and direction.

In choosing a projection, we seek to minimize the distortion in one or more of the above properties, depending on which aspect is more important. **Conformal** projections maintain local angles and shapes, **equal-area** projections maintain area, **equidistant projections** maintain distance, and **true-direction** maps maintain directions with respect to a fixed central point.

No projection maintains all of the above characteristics, but some projections maintain more than one. For example, the Universal Transverse Mercator (UTM) projection, commonly used in the United States, is

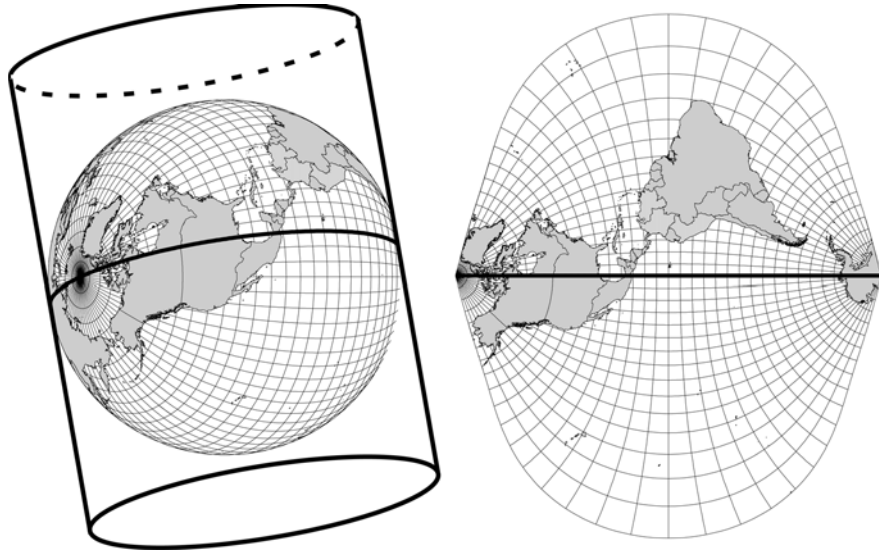


Figure 10-2

The UTM projection of the earth's surface, with a central meridian through Dodge City, Kansas (100°W).

designed to be both conformal and equal area. Note in Figure 10-2 how the projection has the least distortion along the centerline (central meridian). In order to minimize distortion and maintain equal area and conformity, UTM projections are not used more than 3 degrees longitude away from the central meridian. Because of this, the earth is divided into 60 UTM zones. The continental United States spans UTM Zones 10 through 19. The coordinate in the east-west direction is 500,000 at the center of each zone. The north-south UTM coordinate is zero at the equator and is measured northward. Because the coordinate system is referenced to the center of each zone, i.e., the x -coordinate starts over in each zone, features may not be mapped across UTM zones without reprojection into a common coordinate system or projection.

Even though maps may be in the same coordinate system, each can have a different datum, depending on when the data were compiled. However, if data are in the same coordinate system (geographic or Cartesian) and on the same datum, they should overlie. Some discrepancy may still exist with data from different sources because the scale and spatial detail in one dataset are different from those in another. Figure 10-3 shows two streams overlaid on top of a digital orthophoto, where an overlay problem clearly exists. An **orthophoto** is a photograph that has been rectified to a georeferenced coordinate system and can be correctly positioned on a map. The illustrated orthophoto was registered to a coordinate system based on the

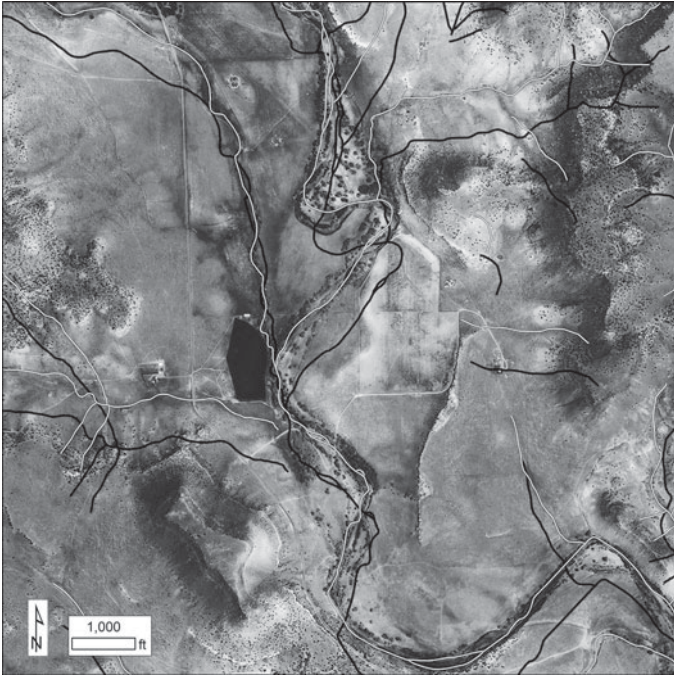


Figure 10-3

Two streams shown on an digital orthophoto showing hydrographic features with mismatched datum (Black stream, NAD27; white stream, NAD83).

NAD83 datum. The second stream, displaced to the south and east, was registered to the same coordinate system, but on an older datum, NAD27. Hydrography compiled in the same coordinate system based on NAD83 (shown with black dashed lines) matches well with stream channel features in the photograph. The NAD27 stream (shown in white) is inconsistent with the photograph. This type of error is referred to as misregistration. Misregistration may be particularly important when map features are used in hydraulic computations such as floodplain delineation. Conversion routines exist to transform spatial data from one datum to another within most GIS software systems. Significant care should be taken to correctly enter projection data when using these transformation routines.

Figure 10-4 shows the earth projected onto a plane tangent to the North Pole. This projection, called the **stereographic** projection, has long been used for navigational purposes and has been used more recently for hydrologic purposes. Vieux and Thompson (2004), Vieux (2004), and Snyder (1987) give information on the datum, projection, and coordinate systems used in GIS and geospatial data.

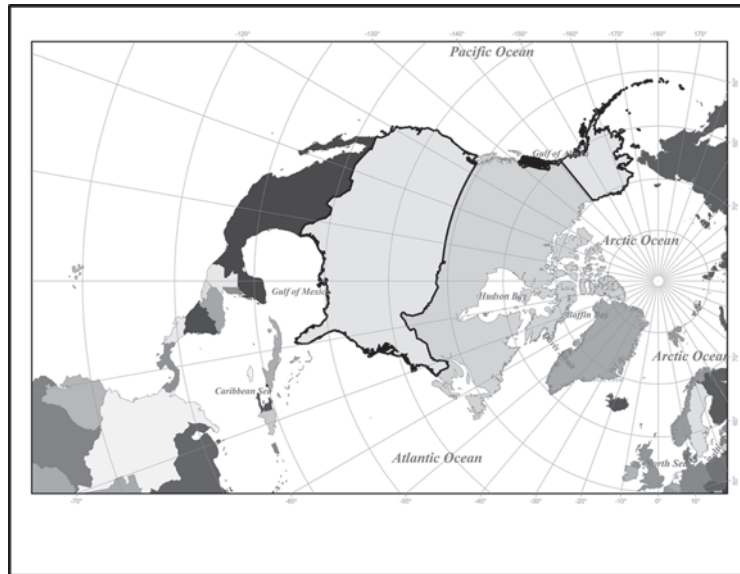


Figure 10-4

Stereographic polar projection showing countries together with lines of longitude and latitude.

10.3 DIGITAL REPRESENTATION HYDROLOGIC PARAMETERS

Parameters of importance to hydrologists that can be derived from digital data sources include rainfall, infiltration, evapotranspiration, and hydraulic roughness. Rainfall may be represented as a time series at a point measured at a gage, as a radial-polar array or a gridded array of rainfall rates derived from radar or isohyetal contours. Infiltration rates, derived from soil maps, are generalized over the polygon describing the soil-mapping unit. Land use/cover may be used to develop evapotranspiration rates or estimates of hydraulic roughness from polygonal areas or from a raster array of remotely sensed surrogate measures.

A dataset may be based on either a direct measure of the physical characteristics or an indirect (surrogate) measure requiring conversion or interpretation. An example of an indirect measurement is NEXRAD's estimation of rainfall from radar reflectivity. Chapter 11 has a more detailed discussion of using NEXRAD rainfall for hydrologic applications.

Soil Type

Computation of direct runoff requires an estimate of infiltration characteristics for the different soil types in a drainage area. Soil-type data are widely available for the United States and can be downloaded from the Natural

Resources Conservation Service at www.nrcs.usda.gov. A **soil-mapping unit** is the smallest unit on a soil map that can be assigned a set of representative properties. Soil maps and the associated soil properties form a major source of data for estimating infiltration. Some adjustment is inevitable when estimating infiltration parameters from generalized soils databases for hydrologic modeling. Obtaining infiltration parameters from soil properties requires reclassification of the soil-mapping unit into a parameter meaningful to the hydrologic model.

Land Use/Cover

The land surface condition can affect hydrologic processes such as runoff and evapotranspiration. A GIS map of the land use and associated cover is an important descriptor of the hydrology, whether it is vegetative, such as pasture or forest, or the surface is modified with asphalt or concrete pavement. Land use and land cover (LULC) GIS data are compiled for many applications and may be useful for hydrologic modeling. Digital LULC maps, derived from direct observation or remotely sensed data or digitized from paper LULC classification maps, can be used to develop hydrologic parameters such as hydraulic roughness, albedo, or aerodynamic roughness height, which influences evapotranspiration.

General-purpose LULC maps may or may not contain sufficient detail or classification to be useful for deriving hydrologic model parameters. LULC maps and the classification scheme are generally not developed for hydrologic purposes. For example, in Figure 10–5 there is discrimination between pasture and cropland; however, the Anderson classification scheme used in USGS LULC maps makes no distinction between these two categories (Table 10–1). In terms of runoff rate or evapotranspiration, these two categories may behave differently. In any case, the hydrologist must exercise judgment when interpreting generalized land use categories for purposes of hydrologic modeling.

Hydraulic roughness is a parameter that controls the rate of runoff over the land surface. Classification codes from the Anderson classification scheme are listed in Table 10–1 with second-level categories indented below the major categories. Maps of hydraulic roughness can be derived using a lookup table that relates hydraulic roughness (see for example, Table 4–2) to the LULC classification scheme (Vieux, 2004). Simple reclassification of a LULC map into hydraulic roughness represents the deterministic variation. Other sources of LULC may include remotely sensed data. In either case, a lookup table transforms the LULC into a map of hydraulic roughness parameters. On-site experience and published values are helpful in establishing at least an initial map for calibration of distributed or lumped parameter models.

Reclassifying the National Land Cover Database (NLCD) dominant land use/cover into hydraulic roughness assumes a lookup table that is

Table 10–1. Anderson Land Use Land Cover Classification Codes: First- and Second-Level Categories

1. Urban or Built-Up Land	6. Wetland
11 Residential	61 Forested Wetlands
12 Commercial Services	62 Nonforested Wetlands
13 Industrial	7. Barren Land
14 Transportation, Communications	71 Dry Salt Flats
15 Industrial and Commercial	72 Beaches
16 Mixed Urban or Built-Up Land	73 Sandy Areas Other than Beaches
17 Other Urban or Built-Up Land	74 Bare Exposed Rock
2. Agricultural Land	75 Strip Mines, Quarries, and Gravel Pits
21 Cropland and Pasture	76 Transitional Areas
22 Orchards, Groves, Vineyards, Nurseries	77 Mixed Barren Land
23 Confined Feeding Operations	8. Tundra
24 Other Agricultural Land	81 Shrub and Brush Tundra
3. Rangeland	82 Herbaceous Tundra
31 Herbaceous Rangeland	83 Bare Ground
32 Shrub and Brush Rangeland	84 Wet Tundra
33 Mixed Rangeland	85 Mixed Tundra
4. Forest Land	9. Perennial Snow and Ice
41 Deciduous Forest Land	91 Perennial Snowfields
42 Evergreen Forest Land	92 Glaciers
43 Mixed Forest Land	
5. Water	
51 Streams and Canals	
52 Lakes	
53 Reservoirs	
54 Bays and Estuaries	

based on roughness that is characteristic of a particular land use/cover classification, but should be guided by local hydrologic knowledge of the area. Assignment of hydraulic roughness to land use/cover classified from remotely sensed data has been compiled by Engman (1986). From the NLCD geospatial data, impervious cover can also be estimated from Landsat imagery. A map of fractional impervious area is seen in Figure 10–5(a) that was derived from the NLCD landcover data. In this dataset, no distinction is made with regard to directly connected impervious area (DCIA) that may affect stormwater runoff; however, the NLCD maps of impervious cover can provide a starting point for assigning infiltration parameters to an urbanized or partially urbanized watershed. An example of a hydraulic roughness map based on reclassification of the NLCD land use/cover is shown in Figure 10–5(b). This map is used in Example 11–4 in Chapter 11 for demonstration of distributed hydrologic modeling. The NLCD land use/cover classifications are presented in Table 10–1 and can be used to assign roughness based on lookup values established from local knowledge of the watershed. Overland roughness derived from remotely

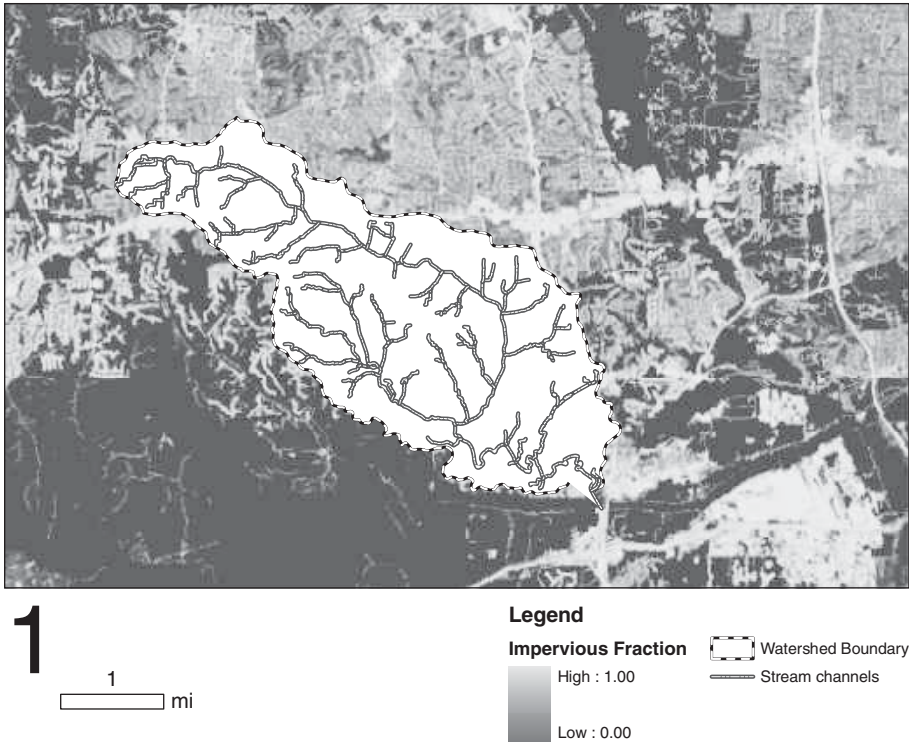


Figure 10-5(a)

Impervious cover derived for the watershed from the NLCD remotely sensed data.

sensed data may not be representative of channel areas particularly if channels are left in natural conditions, e.g., rough surfaces, while overland areas comprise smoother man-made surfaces. If urbanization, wildfires, or other modification of the vegetative cover have taken place since the compilation of the NLCD (e.g. 2001), then adjustment should be made to the roughness values.

The USGS provides these datasets and associated maps as a part of its National Mapping Program (see the Online Internet Resources link for the USGS Seamless Data Distribution System at the end of this chapter). In Chapter 4, Table 4-2 presents typical Manning hydraulic roughness values for various cover types that can be assigned to the Anderson LULC classifications, but with hydrologic judgment. Example 10-2, “Deriving Hydrologic Parameters from GIS Maps of Land Use and Soils,” at the end of this chapter describes the process of reclassification of land use from generalized categories to hydraulic parameters controlling overland flow and infiltration.

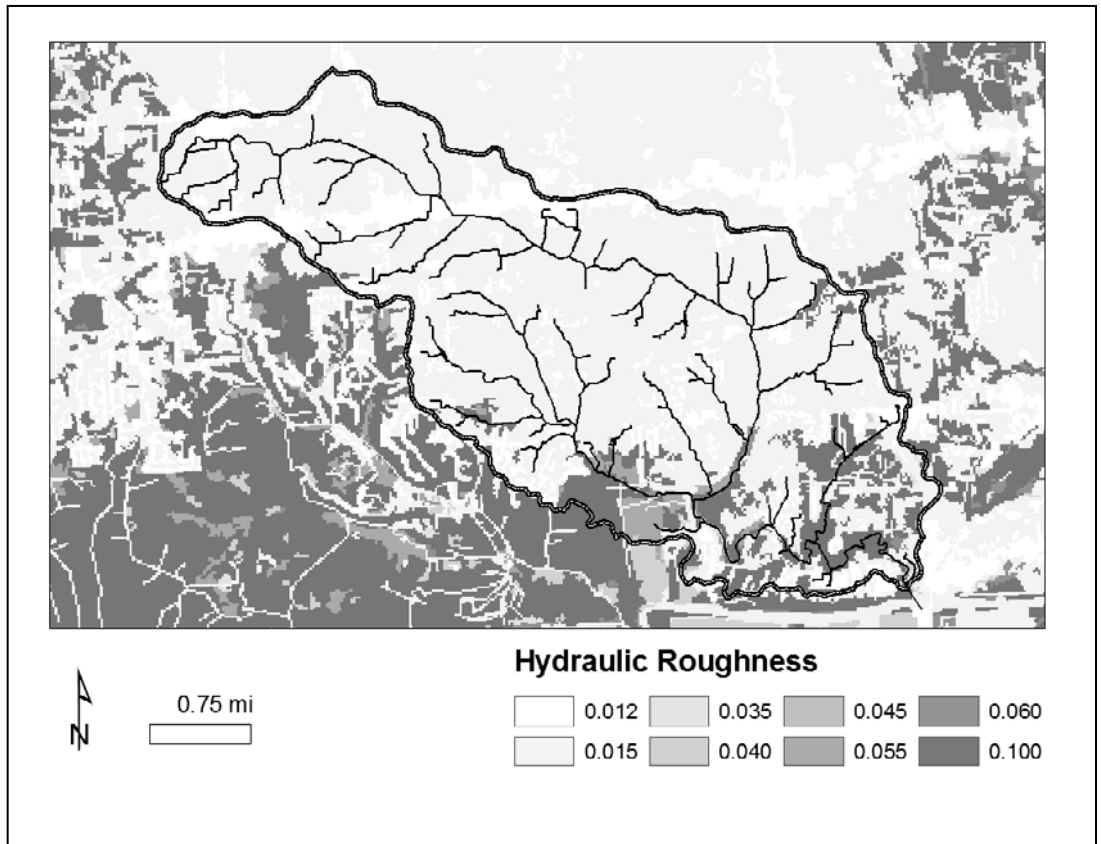


Figure 10-5(b)

Hydraulic roughness interpreted from land use/cover.

10.4 DIGITAL REPRESENTATION OF TOPOGRAPHY

Topography plays an important role in the distribution and flux of water in natural and man-made systems. The automated extraction of topographic parameters from **digital terrain models (DTMs)** using GIS is currently recognized as a viable alternative to traditional surveys and manual evaluation of topographic maps (Moore et al., 1991; Martz and Garbrecht, 1992). A DTM is any digital representation of the terrain, such as a series of point elevations, contour lines, and triangular facets, composing a **triangulated irregular network (TIN)**, also known as a **digital elevation model (DEM)**.

Contours

Many existing paper maps (USGS 7½-minute quadrangle series) use contours to describe the topography. An **elevation contour** is a line connecting the points on the earth's surface having the same elevation. Representation of

a surface using contours shows gradients and relative minima and maxima. Many of the original DEMs produced by the USGS were generated by linear interpolation of digitized contour maps. The USGS elevation contours have intervals and errors that may be larger than those associated with surveyed contours used in civil engineering site development. However, even though the elevations may be uncertain, fail to show topographic detail, and contain vertical/horizontal errors, the contour data can be useful for preliminary delineation of watershed areas and computation of channel slope for hydrologic analysis.

Digital Elevation

One of the most useful types of geospatial data is digital terrain. Useful hydrologic quantities can be derived from a DEM. Other terrain attributes such as slope and drainage direction are derived from a DEM. A model of terrain traditionally relies on contours of equal elevation that define watershed boundaries, channel slope, and other features. Within a GIS, representation of terrain can be accomplished with contours, gridded elevations (also known as a raster DEM), and triangular irregular networks (TIN). Watershed characteristics such as slope and drainage length can be derived from a digital model of the terrain. A **raster DEM** consists of an array of numbers representing the spatial distribution of elevations. It may consist of elevations sampled at discrete points or the average elevation over a specified segment of the landscape—usually the former. Raster DEMs are one of the most widely used elevation data structures because of the ease with which computer algorithms are implemented using grid-based data.

Digital elevation data are acquired through a technique that relies on aerial photography to produce a stereographic image. With survey controls to establish horizontal and vertical position (elevation), contours are produced from the stereographic images. Interpolation of elevation from the contours results in a regularly sampled grid of elevations, which are then used to define watershed drainage areas and other derivative quantities. Recent advances in laser altimetry called light detection and ranging (LiDAR) can achieve submeter resolution. Due to expense, acquisition of LiDAR data is usually reserved for special projects of limited extent, such as floodplain mapping, where high impact or economic value demands high resolution. Due to high sampling frequency of the reflected laser light, many nonterrain features are recorded that must be removed by postprocessing of the data. LiDAR elevation data are collected and processed as a regular array of elevations in raster format. Digital data describing terrain are available for the United States and much of the world from the Internet at 1-km or better resolution, and rounded to the nearest one meter in elevation. The USGS has produced these DEMs at 90-meter resolution for most areas in the United States,

with 30-meter data commonly available and even 10-meter resolution in some U.S. locations (see Internet Resources at the end of this chapter). While generally available DEM data are useful for analysis of larger watersheds, they may be too coarse for small drainage areas, urbanized areas, or flat terrains that lack sufficient vertical resolution, i.e., elevation difference for reliable delineation.

A raster DEM can be used to delineate efficiently watershed boundaries, stream networks, and hydrologic characteristics of a watershed. Many of the methods used for watershed delineation stem from Peucker and Douglas (1975), O'Callaghan and Mark (1984), and Jenson and Domingue (1988). Many of the innovations since these pioneering methods deal with difficulties posed by pits, anomalies, and flat areas within the digital representation of terrain. While a raster DEM can produce accurate boundaries, additional processing is usually required to force the boundary to agree with accepted or known boundaries. Errors in automatically generated boundaries can stem from the precision of the elevation model among other factors (Fisher and Tate, 2006), which may not be sufficient to pick up boundaries in flat areas. Various methods have been developed for overcoming the difficulty with flat areas when delineating a watershed from digital terrain data that may contain pits or anomalies. Zhu et al. (2005) developed a hybrid method for depression filling and scanning neighboring cells for assigning flow direction over flat areas. The relationship between the drainage network and hydrologic response is termed geomorphic. Moussa (2003) compared and classified the basin shape and delineated stream network in an effort to identify scale effects present in the relationship between morphometric properties and the basin hydrological response. Selecting a threshold that defines a stream channel will result in a more or less detailed stream channel network that should correspond to actual watershed geomorphic characteristics. Methods for automatic delineation of watershed boundaries from DEMs are discussed in Section 10.5.

Triangulated Irregular Network

A **triangulated irregular network (TIN)** represents a surface as a set of nonoverlapping contiguous triangular facets of varying size and shape. Generally, a TIN is produced from a DEM through GIS software and has several distinct advantages over contour and raster representations of surfaces. The primary advantage is that the size of each triangle may be varied such that broad flat areas are covered with a few large triangles, while highly variable or steeply sloping areas are covered with many smaller triangles. This provides some storage and detail efficiency over raster data structures, since the element may vary in size according to the variability of the surface. TINs have become increasingly popular because of their efficiency in storing data and their simple data structure for accommodating irregularly spaced elevation data. Most TIN models assume planar

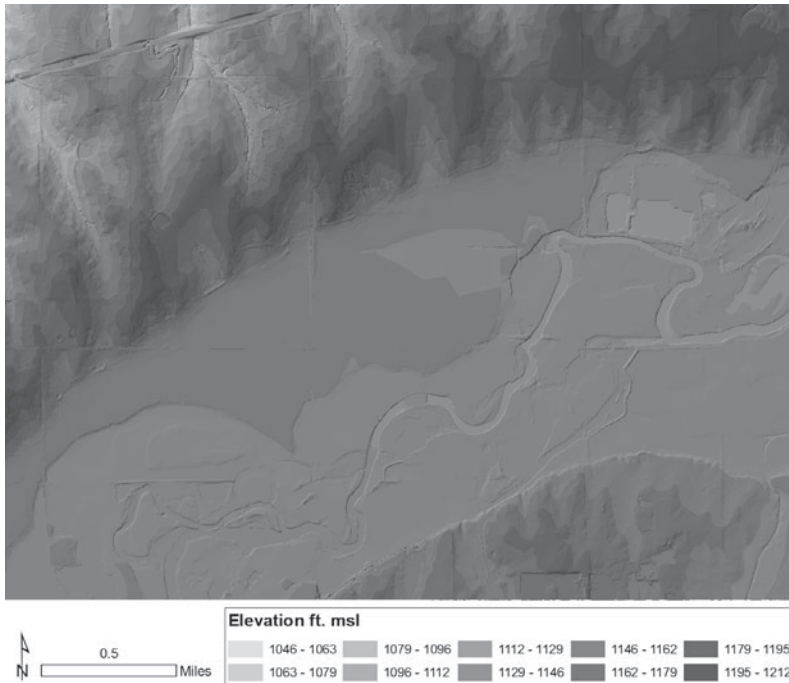


Figure 10–6
TIN elevation model derived from a contours.

triangular facets for the purpose of simpler interpolation or contouring. Vertices in TINs describe nodal terrain features (e.g., peaks, pits, or passes), while edges depict linear terrain features (e.g., break, ridge, or channel lines).

Depending on the usage or software requirements, it may be necessary to convert a DTM from one type (e.g., DEM) to a different type (e.g., TIN). Building a TIN from a DEM requires some automated method for selecting which DEM points will become TIN vertices. Figure 10–6 is a TIN created from a grid DEM. Notice the larger triangles representing flat areas and the smaller, more numerous, triangles in areas where there is more topographic relief. Because DEMs are widely available at a relatively low cost, they are often the basis for creating TINs (Lee, 1991).

GIS processing of digital data is useful for both lumped parameter and distributed models. A lumped parameter model, such as HEC-HMS, assumes that hydrologic parameters, such as percent imperviousness or land use, can be approximated by a single lumped subwatershed value. A distributed

model uses a gridded representation of all hydrologic parameters (e.g., elevation, soil type, imperviousness) to perform model calculations, usually over a grid system. In either case, delineated watersheds are often required for hydrologic analysis.

Watershed Delineation Using Digital Terrain Models

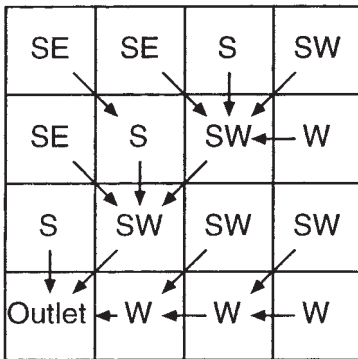
Stream and watershed delineation from DEMs are presented by Olivera (2001). Algorithms for the calculation of stream segment and watershed hydrologic parameters are also discussed. These parameters are curve number, area, and lag time for the watersheds, and routing model and travel time for the stream segments. A watershed, with its subwatersheds and streams, is the basic structure for hydrologic computation for both lumped parameter and distributed hydrologic models. Moore et al. (1991) discuss the major data structures for watershed delineation, including the grid, TIN, and contour methods. Methods for delineation of watersheds using digital terrain data for hydrologic applications are presented in Tarboton (1997) and Turcotte et al. (2001).

Watershed boundaries are primarily delineated in a GIS using a DEM. Within the GIS algorithm used for delineation, the steepest slope determines the direction of flow from each cell. GIS algorithms can use this information to determine how many cells flow into each grid cell, resulting in a delineated watershed and stream network (Figs. 10–7 and 10–8). The basic technique for delineation of a watershed is to consider a 3×3 kernel of cells. The direction of flow from a given cell to its neighbor is found by comparing the elevation difference between the central cell and its surrounding cells. Figure 10–7 shows a simplified example of how the eight possible directions of flow are found. In Figure 10–7(a), the elevation of each cell is compared to the neighboring cells in the eight possible directions: E-W, N-S, and diagonal directions. In Figure 10–7(b), the drainage direction in each cell is assigned. In the final step, the stream network is defined as seen in Figure 10–7(c), starting with the cell designated by the value 4, which indicates the flow accumulation. From this delineation process, flow direction, flow accumulation, and stream network are derived from a DEM. Applying this process to the entire watershed, i.e., considering all kernels, the drainage network is determined as shown in Figure 10–8.

As required for most hydrologic modeling applications, automated watershed and stream network delineation seeks to produce a fully connected drainage network of single cell widths. Drainage networks produced by automated methods may not be very accurate in flat areas. Because elevations in many DEMs are rounded off, drainage directions and the watershed boundary may be in error. If a high-quality map of the stream network and watershed boundary is available in vector format, they may be used to constrain the automated watershed delineation. A watershed and stream vector map may be used to *burn in* the elevations, forcing

60	60	59	58
61	58	56	58
56	54	57	57
50	52	54	56

(a)



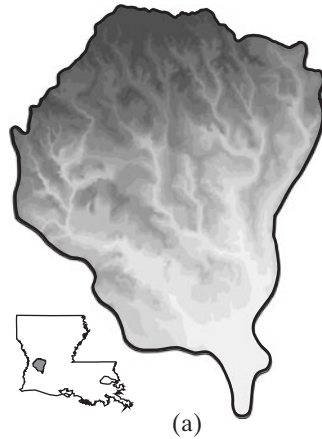
(b)

0	0	0	0
0	1	4	0
0	8	0	0
15	4	2	0

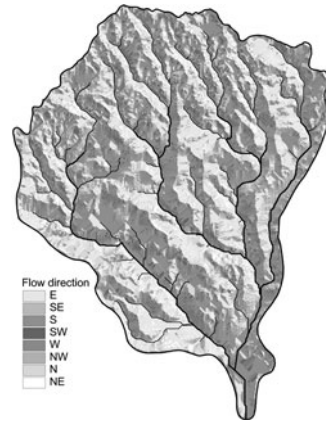
(c)

Figure 10-7

Schematic representation of the eight-direction pour-point method. (a) Flow direction grid. (b) Flow accumulation grid. (c) Stream network.



(a)



(b)



(c)

Figure 10-8

Results of applying the eight-direction pour-point method to the Whisky-Chitto watershed in Louisiana. (a) Flow direction grid. (b) Flow accumulation grid. (c) Stream network.

the drainage network to coincide with the vector map depicting the desired drainage network. By burning in the stream (i.e., artificially lowering the DEM at the location of mapped streams), the correct location of the automatically delineated watershed and corresponding stream network is preserved. Watersheds and stream networks delineated from a DEM become a structure for organizing lumped and distributed hydrologic model computations. Once the watershed and stream network have been delineated using automatic methods, the resulting map should be verified in the field and compared with other maps compiled from other sources. Figure 10–9(a) shows the effect of not burning in a stream in a watershed that was automatically delineated from 1-m resolution LiDAR. The vector representation of the main channels is indicated by the continuous black line, whereas the gray elemental flow paths within a 75-m resolution model grid in *Vflo*TM deviate as indicated within the dotted ellipse. Once the stream channel is burned in, i.e., delineation is constrained to follow the channel, the delineated flow paths conform closely to the stream channel. No longer does the stream channel *jump* over into an adjacent catchment as seen in Figure 10–9(b), and the delineated flow paths follow the stream channel defined by the vector map.

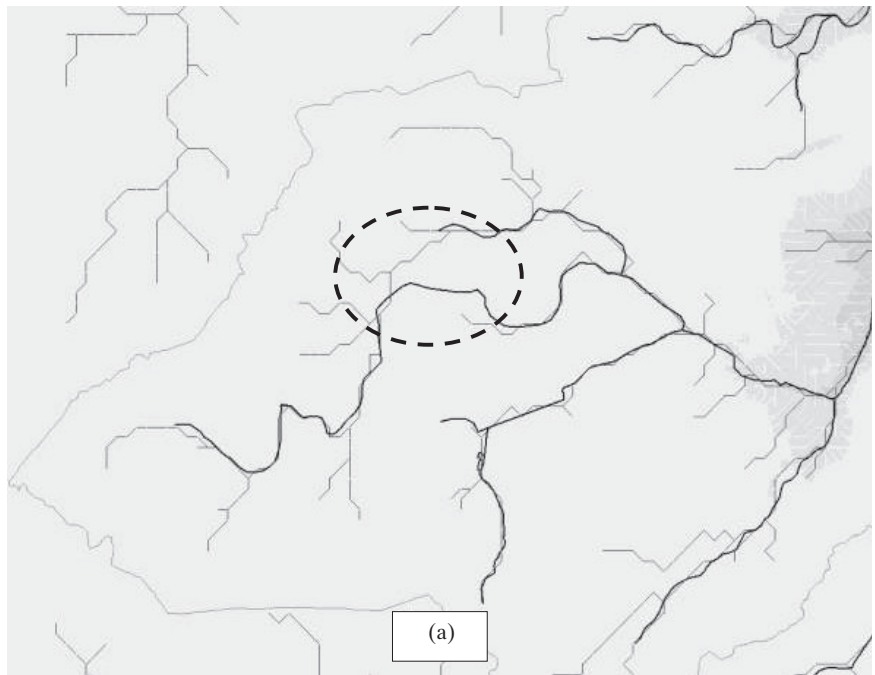


Figure 10–9(a)

Watershed delineation without stream channel enforcement.

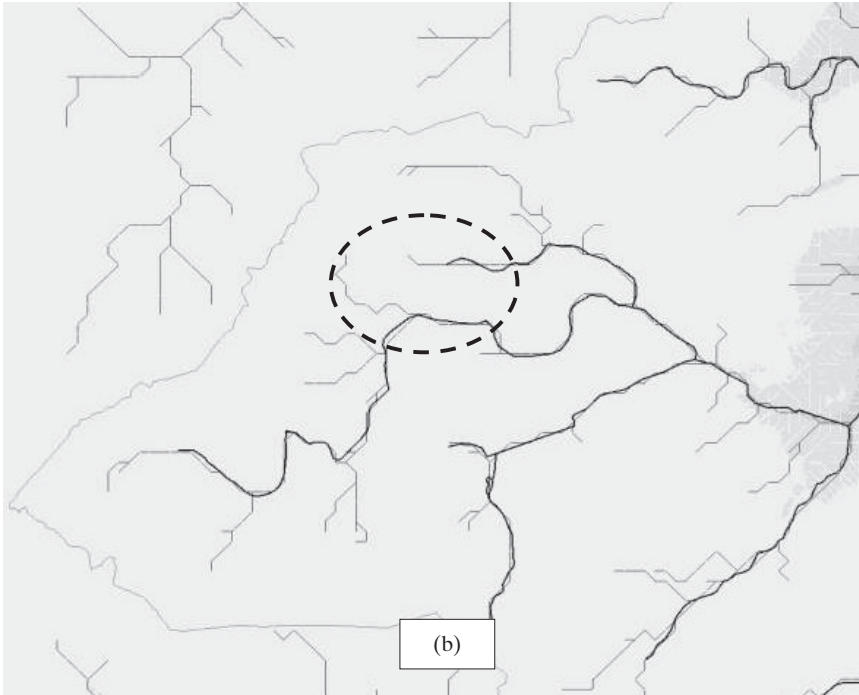


Figure 10-9(b)
Watershed delineation with stream channel enforcement.

A watershed boundary can be established using a vector boundary to form a wall to constrain the automated delineation. A wall confines the automated process so that the delineation conforms to the accepted boundary of the watershed. Without constraint, the elevation in each grid may allow the delineated watershed to deviate from the boundary due to elevation errors or flatness of the terrain. Constraining the automated watershed delineation is demonstrated by the results from the input processor in *Vflo*TM called Auto-BOP (Automated Basin Overland Properties) shown in Figures 10-10(a) and 10-10(b). In Figure 10-10(a), delineating the watershed without the use of a vector boundary allows flow paths to extend across the vector boundary and into an adjacent watershed (see ellipse and arrow). The watershed delineation in Figure 10-10(b) is consistent with the vector boundary because it is used as a *wall* in the process. The dotted ellipses in Figures 10-10(a) and (b) indicate the region where flat slopes along the divide or urbanized infrastructure resulted in drainage area capture. While digital terrain data can be invaluable for delineating watersheds based on natural elevations, urbanization can alter the pathways especially where constructed drainage infrastructure conveys runoff in directions other than that dictated by the steepest decent. The supplemental website contains the presentation that describes the procedure.

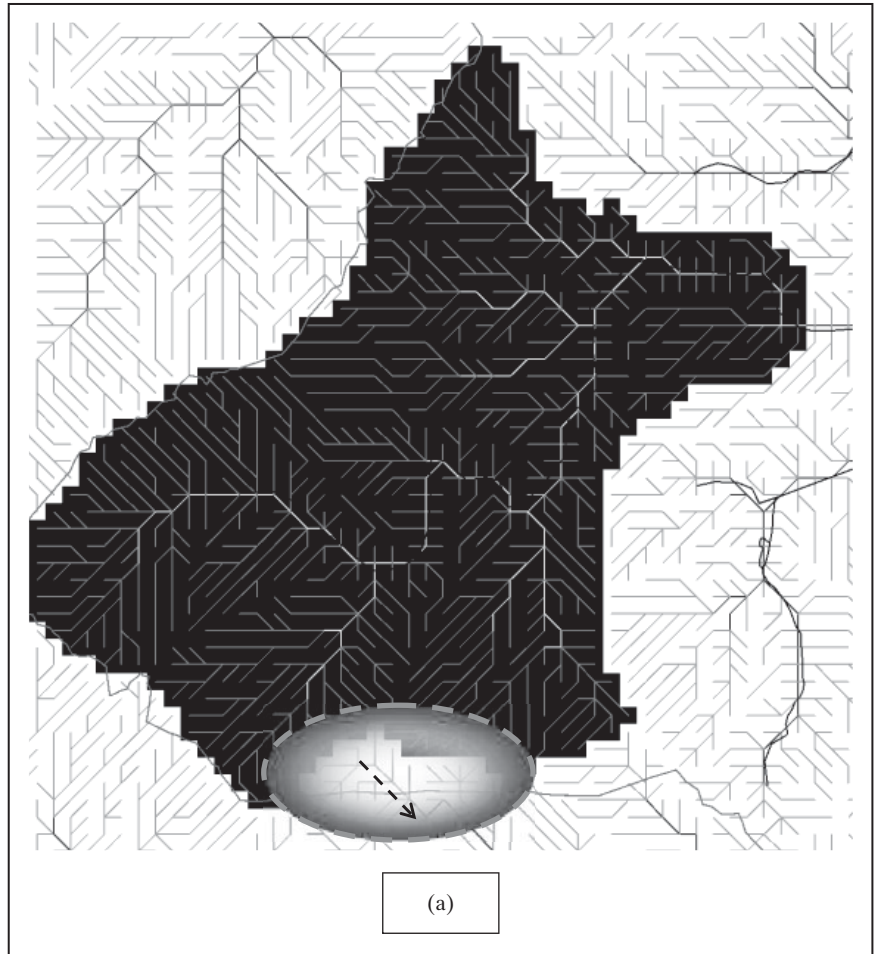


Figure 10-10(a)

Watershed delineation without enforcement of the watershed boundary.

GIS Tools for Hydrologic and Hydraulic Modeling

One of the main tasks in creating and applying a hydrologic or hydraulic model is the preparation of the appropriate datasets for modeling a watershed or basin. Early models were data-intensive, and most of the parameters required for a model have to be estimated manually and typed into computer files (i.e., FORTRAN input files), using very specific formats. As a result, hydrologists have to be specially trained in the application of these models. Modern computer-based technologies such as spreadsheets, databases, and graphical capabilities greatly facilitate some of the time-intensive data entry procedures. User-friendly interfaces allow hydrologists to enter

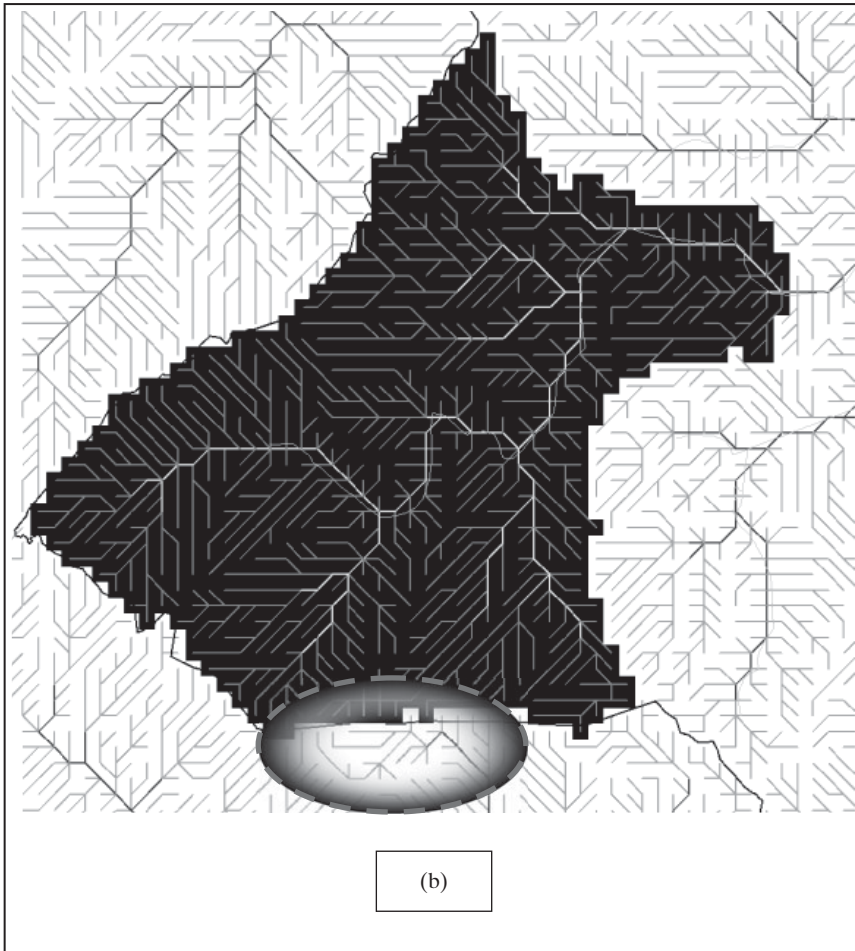


Figure 10–10(b)
Watershed delineation with enforcement of the watershed boundary.

model data in free formats and provide online help screens to assist the user in understanding the specific parameters required. There are two main GIS modules for use with the lumped hydrologic model: the HEC-HMS and HEC-RAS hydraulic model (see Chapters 5 and 7). HEC-GeoHMS and HEC-GeoRAS are modules that run with the ESRI software, ArcView and ArcGIS. Besides the user manuals for this software mentioned in Chapters 5 and 7, additional information on extracting hydraulic information from digital terrain is provided by Ackerman et al. (2000) and by Tate et al. (2002), who describe the formation of a digital terrain model for floodplain mapping.

EXAMPLE 10–1**GIS PROCESSING OF DIGITAL TERRAIN**

A basic application of GIS in hydrology is the delineation of watershed boundaries and corresponding stream network. Several modules are available for automatic delineation from a DEM. Because a vector map of the watershed boundary or stream channel location may have more accuracy than the DEM, the automated delineation can be improved using the vector maps to constrain the delineated boundary and stream channel location. Constrained watershed delineation helps ensure that the resulting maps are consistent with the watershed boundary and stream channel mapped in vector format. This example provides an overview of how to *burn in* a DEM with a stream network and use the watershed boundary to constrain the watershed delineation. The watershed delineation relies on the Arc Hydro Tools extension in ArcMap. The watershed delineation is performed using LiDAR data at 4.7-meter resolution that is resampled to coarser resolution. Detailed instructions and presentation material called *GIS Processing of Digital Terrain* may be found on the textbook website along with sample data (see Appendix E).

The steps followed in the presentation are:

- Reproject original DEM to the desired projection with horizontal and vertical units of meters.
- Resample original DEM to a selected resolution.
- Merge stream network and basin/subbasin outline into a DEM to ensure proper flow direction.
- Use Arc Hydro Tools extension to produce flow direction, flow accumulation, and a stream grid.

The resulting watershed maps demonstrate the burn-in procedure using a combination of raster and vector map information. The flow direction map is for use in the distributed hydrologic model, *Vflo*TM. An alternative approach is to use the HEC-GeoHMS processing module to process the DEM and to produce subbasin boundaries and stream channel map information for input to HEC-HMS.

EXAMPLE 10–2**DERIVING HYDROLOGIC PARAMETERS FROM GIS MAPS OF LAND USE AND SOILS**

Generalized land use/cover maps are now available for most locations. Such maps may be useful for hydrologic analysis if the mapped categories are representative of current conditions or those present during a particular storm event. To obtain hydraulic or hydrologic parameters from such a map, reclassification is necessary to convert the dominant land use/cover into hydrologic model parameters. Figure 10–5 shows a hydraulic roughness map derived from NLCD land use/cover classifications.

Interpretation of the dominant land use category is needed to express the category in terms of an appropriate value of the corresponding hydraulic roughness parameter. Because a particular land use category may be heavily or lightly vegetated, the assigned hydraulic roughness should be adjusted to represent this in the model. Similarly, soil maps can be reclassified based on soil texture into infiltration parameters for use in the Green–Ampt equation. Detailed instructions and presentation material called *Deriving Hydrologic Parameters from GIS Maps of Land Use and Soils* may be found on the textbook website along with sample data.

The steps followed in the presentation are:

- Reclassify land use/soils datasets to hydrologic parameters defining hydraulic roughness and infiltration.
- Use multiplicative factor when reclassifying integer-based land use/soils classes, then divide out multiplicative factor with Raster Calculator (RC).
- Resample roughness/infiltration values to desired $Vflo^{\text{TM}}$ model grid resolution.
- Clip the resampled grid to a specific model domain.
- Export roughness/infiltration grids and load in $Vflo^{\text{TM}}$ as ArcGIS Grids or ASCII files.

The resulting watershed maps demonstrate the reclassification of GIS maps of land use/cover and soils into hydraulic and hydrologic model parameters. The resulting maps are for use in the distributed hydrologic model, $Vflo^{\text{TM}}$. Alternatively, the same approach to reclassification can be applied to obtain parameters for input to HEC-HMS by averaging the distributed values over each subbasin area within the watershed.

Numerous GIS programs are available that may be adapted for use in hydrologic applications. Table 10–2 summarizes some of the more common ones and is not intended as a comprehensive list. Note that programs tend to change through time as new ones are developed.

These GIS programs and others can be used by hydrologists to develop hydrologic data and perform analyses in the preparation of model input, and for the interpretation and visualization of hydrologic/hydraulic model results.

Additional Resources

Principles of GIS, its background, and application may be found in books by Burrough and McDonnell (1998). Other references that provide the fundamentals of GIS, spatial analysis, and principles include Clarke 2001 and

10.6 COMMON GIS SOFTWARE PROGRAMS

Table 10–2. Common GIS Software Programs

Software	Description	Modules
ArcGIS Desktop	ArcGIS Desktop is a collection of software products used to create, import, edit, query, map, analyze, and publish geographic information. There are four products in the ArcGIS Desktop collection; each adds a higher level of functionality. This software is distributed by ESRI, Redlands, California. (www.esri.com)	ArcGIS Desktop includes ArcView and other products that provide extensive mapping, data use, and analysis along with simple editing and geoprocessing capabilities. The ArcView Extension called Spatial Analyst provides important terrain-processing functions useful for hydrologic applications.
ERDAS	This software is distributed by Earth Resource Data Analysis System (ERDAS) established in 1978. ERDAS is located in Atlanta, Georgia. (www.erdas.com)	Extensive support for remotely sensed data includes unsupervised classification, geometric correction, and cartographic map production tools. ERDAS IMAGINE® suite supports spatial analysis of imagery from aerial photography and satellite remote sensing. Besides ortho-rectification and reclassification of images, support for raster and vector formats and interfaces with other GIS software packages is provided.
IDRISI	A popular software package for teaching and research. Distributed by Clark Labs, Clark University, Worcester, Massachusetts. (www.clarklabs.org)	IDRISI Andes is an integrated GIS and image-processing software solution for the analysis and display of digital spatial information. The IDRISI software offers GIS and image-processing tools for land planning, decision support, and risk analysis, which are included with tools for spatial statistics, surface analysis, and spatial modeling.
Bentley	Bentley provides software dedicated to architects, engineers, constructors, and owner-operators with comprehensive software solutions for sustaining infrastructure. Company headquarters: 685 Stockton Drive, Exton, PA 19341, United States. (www.bentley.com/)	Bentley Map consists of the Bentley geospatial desktop and server products. It is a full-featured 2-D and 3-D GIS designed for organizations that map, plan, design, build, and operate the world's infrastructure. In addition to GIS software, Bentley distributes the CAD software MicroStation.

DeMers (2004). Terrain analysis and the use of raster data formats are described in Wilson and Gallant (2000) and DeMers (2001).

The proliferation of worldwide geospatial data has made it possible to perform GIS analysis for almost any location using generally available data. Such geospatial data have limitations and may not contain sufficient detail for some hydrologic applications. Methods of obtaining publicly available geospatial data, the software used for processing, and examples of use are discussed in Ralston (2004). The science and technology for measuring the earth are described in textbooks such as Wilson and Gallant (2000) or *Geomatics* by Kavanagh (2009). These textbooks provide a clear description of

terrain data and processing techniques, survey data, GPS, GIS, geospatial data, and remote sensing data, emphasizing the measurement accuracy, database management, geospatial data formats, and coordinate systems employed with such data.

Use of GIS for urban planning and development applications is presented by Easa and Chan (2000). Shamsi (2002) describes the use of GIS in urban/engineering application areas of water, wastewater, and stormwater. These authors address urban planning and development applications of GIS, giving information on practical planning and development. Topics address a wide variety of topics that include regional planning, transportation, public utilities, stormwater and waste management, cultural and resources management, environmental assessment, program evaluation and policy analysis, and education.

The modeling technology available for hydrology has advanced along with the advent of geospatial data useful for watershed modeling. Distributed modeling uses geospatial data directly to model the spatially variable watershed characteristics. The report by ASCE (1999) guides the professional in the selection of GIS modules and distributed models for watershed runoff analysis. GIS hydrology analysis software and techniques are presented in addition to geospatial data types and map projections. Important topics covered include the limitations of available data and the integration of watershed hydrological analysis software with GIS software. An appendix is provided in ASCE (1999), which highlights nine different watershed-modeling systems that operate in conjunction with GIS software and geospatial data. The research monograph by Vieux (2004) presents a unified approach to GIS usage in distributed hydrology. It addresses the technology and theoretical background required for distributed modeling, including GIS processing, radar rainfall, and calibration and case studies. The book's second edition includes software and sample data as well as the background, theory, and practical applications of GIS and geospatial data in distributed hydrologic modeling.

Considering the spatial character of various parameters in hydrologic processes, it is not surprising that GIS has become an integral part of modern hydrologic studies. Global digital databases of topography, LULC, and other data types offer unparalleled capabilities for hydrologic modeling throughout the world. Rapid expansion of spatial data in the United States provides the hydrologist with a rich array of data. GIS maps are now readily available that describe topography, LULC, soils, and rainfall. Additionally, meteorological variables such as rainfall can be estimated with radar and entered directly into GIS-based studies. Care must be taken to ensure that the derived hydrologic parameters, delineated basins, projections, and datums are accurate and have sufficient precision to be useful in hydrologic analyses.

SUMMARY

**ONLINE
RESOURCES***Hydrologic Engineering Center, U.S. Army Corps of Engineers*

The Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) is an extension for use with ESRI ArcView. The modules can produce watershed delineations using digital elevation models. The results can be used independently or for input in the HEC-HMS hydrologic model available from the same site.

<http://www.hec.usace.army.mil/software/hec-geohms/>

HEC-GeoRAS is a set of procedures, tools, and utilities for processing geospatial data in ArcGIS using a graphical user interface (GUI). The interface allows the preparation of geometric data for import into HEC-RAS and processes simulation results exported from HEC-RAS.

<http://www.hec.usace.army.mil/software/hec-ras/hec-georas.html>

Minnesota Department of Natural Resources

This site provides online information for using the ESRI ArcView 3.x GIS software and data. The ArcView Utility Tools extension contains a number of extended GIS functions that are commonly used by resource managers but not available on the standard ArcView interface. The ArcView Utility Tools contains a form that contains a number of buttons that perform a variety of GIS processing functions. Some of these are directly related to geographic analysis (clip, buffer, intersect, etc.), and some are general utility commands that have been commonly requested.

<http://www.dnr.state.mn.us/mis/gis/tools/arcview/extensions/tools/tools.html>

FEMA Digital Flood Maps Online

FEMA's Map Service Center (MSC) is the official government source for flood hazard mapping information, products, and services.

<http://msc.fema.gov/>

EPA GIS Information

EPA's National Geospatial Program coordinates the Agency's geospatial data, applications, policies, and programs. This website provides access to data and applications besides related links and is found at:

<http://www.epa.gov/geospatial/>

USGS Seamless Data Viewer and Downloads

The United States Geologic Survey (USGS) maintains the Seamless Data Distribution System (SDDS), which enables the user to view and download many geospatial data layers, such as National Elevation Dataset, National Land Cover Dataset, and High Resolution Orthoimagery.

<http://seamless.usgs.gov>

Geospatial Data Clearinghouse Links

The FGDC provides a one-stop portal and helps coordinate the sharing of geographic data, maps, and online services through an online portal, geodata.gov, which searches metadata held within the NSDI Clearinghouse Network. The portal provides "one-stop" access to all registered geographic information and related online access services within the United States. Geographic data, imagery, applications,

documents, websites, and other resources have been catalogued for discovery in this portal. The portal and related information, data, and services may be found at: <http://www.fgdc.gov/dataandservices>

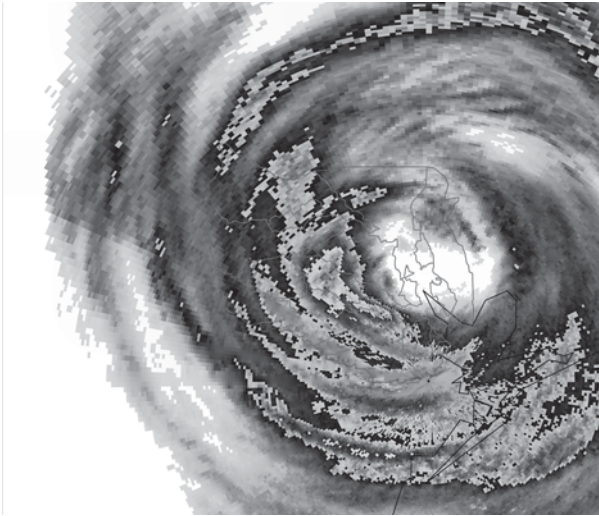
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Chapter 11

Radar Rainfall Applications in Hydrology



Hurricane Ike on September 13, 2008 approaches the Gulf Coast near Houston, Texas. Radar measures reflectivity at high spatial and temporal resolution over watersheds. (Photo courtesy Vieux & Associates, Inc.)

Advances in radar technology have enabled accurate characterization of the distribution of precipitation over a watershed. When properly adjusted with rain gage measurements, radar can provide the spatial and temporal variation of rainfall necessary for hydrologic prediction and analysis. Rain gages provide effective high-resolution temporal information but are limited by sampling at a single point in space. The combination of radar and rain gages can produce representative precipitation measurements over a watershed that is more accurate than can be achieved from either rain gages or radar used alone (Wilson and Brandes, 1979).

When rain gages are the only means of measuring or characterizing rainfall over a watershed, rainfall amounts from several gage measurements must be distributed over the area or assumed uniform over the area. Thiessen polygons or inverse distance weighting are the primary methods for spatially

11.1 INTRODUCTION

distributing rainfall from point measurements. Such spatial distribution methods do not account for the variability of rainfall that occurs between the rain gages. In modeling actual storm events or in continuous simulation, uniform rainfall assumptions can lead to overprediction or inaccurate hydrograph response, which makes radar rainfall attractive. When assumed rainfall distribution patterns are inaccurate or depart from the actual variation of rainfall occurring over a watershed area, model results and hydrologic assumptions can be grossly in error.

Rain gages operated by the National Weather Service (NWS) can provide high-quality data with long record lengths for climatological purposes. Primary gages are located at airports and other permanent installations and record hourly precipitation. Cooperative observers supplement the primary network with observations taken on a daily basis. Even though these data are often the best available, the primary network and cooperative observer networks are generally too sparse to capture the entire spatial variability of precipitation affecting a watershed or river basin. Another source of rain gage information is automated local evaluation in real time (ALERT) sensors, which are installed by local governments and other agencies to monitor rainfall and other variables at higher spatial resolutions than gages operated by the NWS. However, even ALERT and other private gage networks may not be sufficient to capture the spatial variation of rainfall, especially in convective situations where rain-producing storm cells may pass through the gage network without detection of the most intense rainfall rates. Rain gage networks are important sources of rain gage data by themselves and as a means for enhancing the accuracy of precipitation estimates derived from weather radar.

This chapter deals with the use of weather radar in hydrology, with the primary emphasis on distributed modeling of watershed-scale processes. Many studies have shown that accurate measurements of rain intensity, with good spatial and temporal resolution, are essential to hydrologic analysis and prediction. There are three principal methods to measure rainfall: (1) rain gage, (2) radar, and (3) satellite imagery. Besides rainfall, radar estimation of frozen precipitation such as snowfall is not treated here. Radar rainfall, if properly adjusted with rain gage data, accurately characterizes rainfall distribution over a watershed, and temporally as hyetographs. The use of radar for hydrologic purposes is becoming more commonplace for applications where model predictions are used to simulate continuous streamflow or individual events, make flood predictions, or characterize inflow to stormwater, combined, or separate sanitary sewers. Development and application of radar for hydrological purposes is described in a review by Einfalt et al. (2004). Adjustment of radar using rain gages and the factors affecting the accuracy of the rainfall product are described in many references, including Morin et al. (1995), Rosenfeld et al. (1993, 1994), Smith et al. (1996), Sanchez-Diezma et al. (2001), Vieux and Bedient (1998), and Vieux and Vieux (2005).

Rainfall derived from radar fills in the gaps between gage measurements, since radar data far exceed spatial densities of most rain gage networks. While radar provides the spatial and temporal patterns of rainfall, it requires correction or adjustment to remove systematic over- or underestimation of rainfall rates. To understand how radar is used to measure rainfall, one must understand the measurement process, conversion of reflectivity to rain rate, system characteristics affecting resolution and precision of the rainfall data, and bias adjustment for systematic errors. First, we examine the origin and characteristics of radar estimation of rainfall.

The word *radar* is an acronym for radio detection and ranging. Radar was originally developed in the 1930s as a simple device for detecting aircraft. It has evolved to a complex system involving transmitter and receiver components coupled with a computer processing system for detection of a variety of targets. Some of these targets are meteorological in nature and therefore of interest in hydrology. A radar system transmits pulses of electromagnetic energy in the microwave frequency spectrum and then “listens” for the strength of the reflected signal. The timing of the returned signal compared to the transmitted pulse gives the range or distance to an object. The strength of the returned signal, i.e., reflectivity, depends on the cross section of the object and an electrical property called the dielectric constant. The returned signal strength is also termed reflectivity. The Doppler effect measures whether the object is moving away from or toward a radar installation. Relative velocities provide the weather forecaster with valuable information related to the formation of tornadic activity, strong winds, and the motion of storms.

One of the benefits of war-related research and development was the birth of radar meteorology. During World War II, it became obvious that precipitation targets prevented full use of the radar for detection of aircraft. At first, precipitation was an obstacle to the radar’s intended military purpose, but later was modified to improve its ability to detect weather phenomena. Many of the scientific achievements in this area resulted from the availability of military hardware. The NWS and commercial meteorological services have been using radar for more than 30 years to monitor and track a variety of weather conditions, including strong winds, hurricanes, tornados, and precipitation.

Early NWS and Air Force weather radars were converted from airborne systems designed for detecting military objects. Since the introduction of radar technology, several nationwide radar systems have been developed and deployed by the U.S. government, notably the WSR-57 and later the WSR-88D radar network. Successive improvements have been made in microwave technology, sensitivity, spatial and temporal resolution, and the computer systems necessary to process the data into quantities relevant to weather surveillance.

NEXRAD (next-generation radar) was prototyped by the National Severe Storms Laboratory (NSSL) in Norman, Oklahoma. Operation and

Figure 11-1
NEXRAD (WSR-88D)
radar installation
Norman, Oklahoma.



support are provided by the NEXRAD agencies, which are the NWS, the Federal Aviation Administration (FAA), and the Air Force. NEXRAD is technically known as the WSR-88D (weather surveillance radar prototyped in 1988; the D is for Doppler). WSR-88D is a Doppler radar with a 10-cm-wavelength (S-band) transmitter/receiver that records reflectivity, radial Doppler velocity, and the spectrum width of reflected signals. A typical WSR-88D installation is shown in Figure 11-1. Located near Norman, Oklahoma, the radar installation designated by the call sign KTLX was the first operational WSR-88D radar installed in the United States.

Approximately 160 WSR-88D radars have been installed in the continental United States (CONUS) and abroad at U.S. military installations. Figure 11-2 shows the distribution of WSR-88D radars nationwide along with 230-km range circles centered over each radar installation. Coverage of the United States is generally good except in western areas, where few radars exist or terrain causes blockage. Within the continental United States, there is usually at least one WSR-88D radar within 230-km range of a given location. From the strength of the returned signal, and after adjusting for distance and other effects, certain characteristics can be deduced about the target. Water in the form of raindrops, hail, or snowflakes reflects energy transmitted by the radar. In the case of raindrops, the strength of the returned signal is related to the rainfall rate. The primary use of radar in hydrology is the estimation of precipitation based on reflectivity.

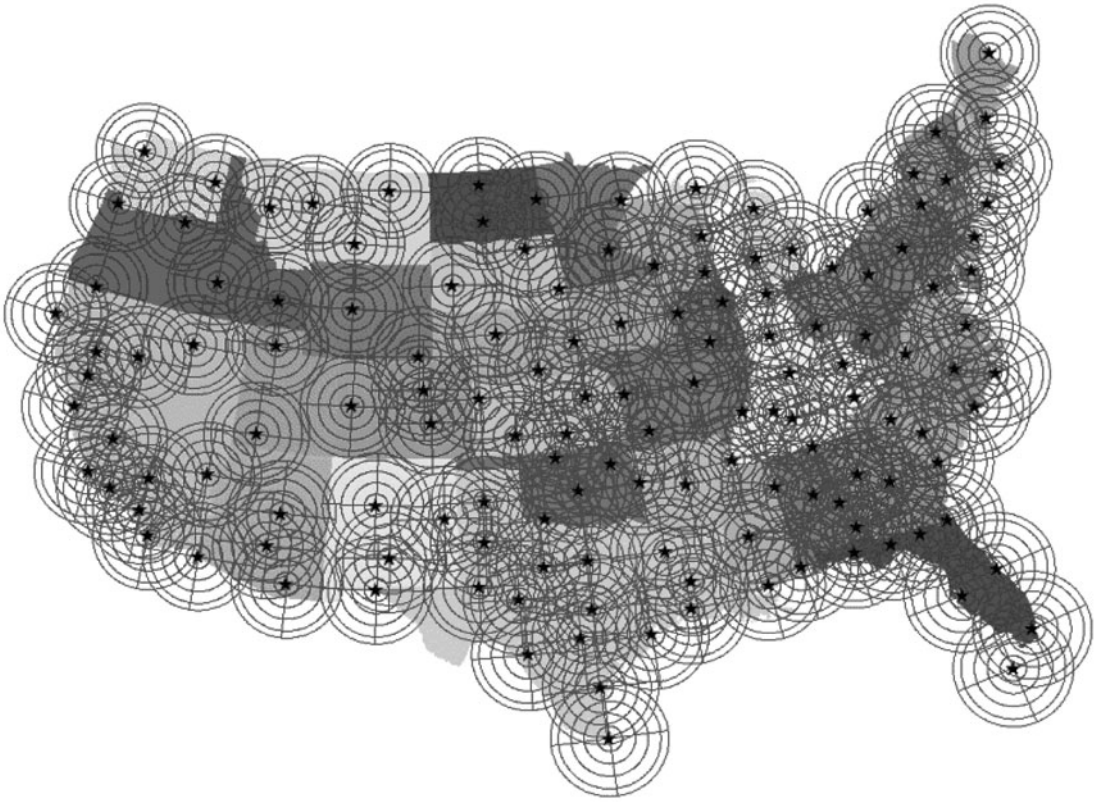


Figure 11-2

CONUS distribution of NEXRAD (WSR-88D) radars nationwide along with 230-km range circles centered over each radar installation.

Enhancement to radar technology through polarization of the radar signal improves discrimination of precipitation phase, i.e., frozen or liquid hydrometeors. Dual polarization, in the vertical and horizontal planes, produces additional measures, called polarization diversity measurements, which are related to rainfall rate and outperform precipitation estimation based solely on single-polarized reflectivity (Doviak and Zrnic, 1993). Ryzhkov et al. (2005a, b) have documented significant improvements in rainfall estimation that can be achieved through the implementation of dual-polarization technology. These and other recently published comparisons compare precipitation estimates between horizontal-polarization Z-R estimates and multiparameter dual-polarization estimates of rainfall. Because of the improved rainfall estimation obtained with dual polarization, the CONUS WSR-88D radars will be upgraded during 2009–2012, depending on federal funding.

Figure 11–3(a)
Cumulus clouds
during a summer
storm near Okla-
homa City, June 16,
2000, at 9:00 P.M.
(Vieux, 2004.)



Cumulus clouds rising over Oklahoma in Figure 11–3(a) were photographed at approximately 9:00 P.M., June 16, 2000. The corresponding radar reflectivity at 8:57 P.M. is shown in Figure 11–3(b). Reflectivity measured by the radar depends on the distribution of raindrop sizes in the clouds, shown in Figure 11–3(a); higher reflectivity (>35 dBZ), shown in black, represents more intense rainfall. Radar is an important source of spatially and temporally distributed rainfall data for hydrologic modeling. Radar is capable of

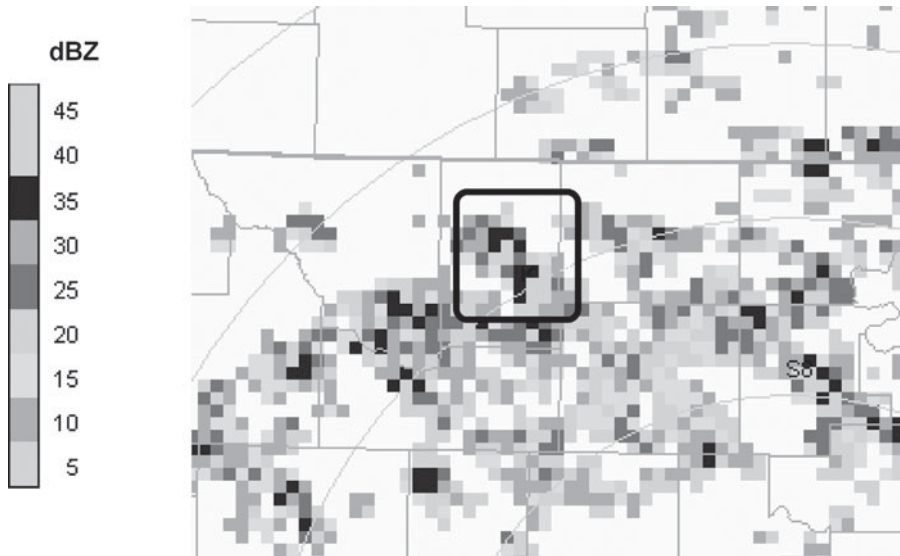


Figure 11–3(b)
Composite reflectivity corresponding to the same time period. The arrow indicates the direction from which the photograph in Figure 11–3(a) was taken. (Vieux, 2004.)

measuring patterns of rainfall not only in space, but also over time. At each revolution of the radar, an updated pattern of rainfall results. Taken as a series in time, the evolution of the storm is mapped as it intensifies or decays, and approaches, passes over, and leaves the watershed of interest. Archival radar data can be used to reconstruct continuous periods or storm events for analysis and hydrologic model calibration. Because the rainfall derived from radar is more representative of actual rainfall patterns, hydrologic model calibration can be more efficient and produces results that are more consistent than those obtained with only a few rain gages in or near the catchment. Using a calibrated model for design carries more confidence, because the model can match an individual event or series of events characterized by radar. Vieux and Bedient (2004) describe the reconstruction of major flood events using archival radar data adjusted to ALERT rain gages to represent the distribution of rainfall over an urban catchment. While flood prediction depends on both terrestrial parameters and rainfall inputs, a key determinant of success is the accuracy of the rainfall and its representativeness over the watershed (Looper et al., 2009; Looper and Vieux, 2011). The accuracy of gage-adjusted radar rainfall estimates for urban sanitary and storm sewer modeling is reviewed in Vieux and Vieux (2005). For radar rainfall usage and processing protocols for hydrologic and hydraulic model calibration applications, refer to Burgess et al. (2003) and Vieux and Bedient (2004).

Radar estimation of rainfall offers unique advantages over rain gages, the most evident of which are (1) coverage over large areas, (2) temporal updates as short as 5 minutes, (3) long-range coverage, and (4) a high resolution in space of 1 degree by 1 km. The frequency band of the radar and its transmitter and receiver and antenna characteristics affect the range and resolution achievable by weather radar. The WSR-88D radar is an S-band radar with a wavelength of 10 cm, which is resistant to severe attenuation by intense precipitation. Other radars include shorter wavelength radars used for broadcast meteorology or tailored systems for specific applications. Within the NWS weather and river forecast offices and for users outside of the federal government, the WSR-88D system serves a wide range of hydro-meteorological applications. Hydrologists are interested in how much rain fell, is falling, or is projected to fall over a particular watershed, which is the primary application of WSR-88D radar precipitation estimates in hydrology. Understanding how radar estimates precipitation is important for effective application of this observational system to hydrology as described below.

11.2 RADAR ESTIMATION OF RAINFALL

Precipitation Detection

Radar transmits pulses of microwave energy and then “listens” for the returned pulse. The power of the received pulse reflected by the target is known as reflectivity. The detection of precipitation targets is dependent on a variety of factors, the most important being (1) atmospheric conditions

between the radar and target, (2) distance from the radar to the target, (3) target characteristics, and (4) radar characteristics.

Atmospheric conditions between the radar and target affect the efficiency with which microwave energy is reflected back to the radar receiver. Backscattering and absorption of the radar signal by the raindrops in the intervening atmosphere reduce, or attenuate, the signal strength. With distance, signal strength is further attenuated, making accurate rainfall-rate estimation difficult. The radar operator does not generally know the size of the raindrops or the number of drops of each size. The probability distribution of drop sizes is known as the drop-size distribution (DSD)—which affects the strength of the returned signal. It is important to realize that radar measures reflectivity at heights extending to 15 km in the atmosphere, below which liquid and frozen hydrometeors may coexist. In tropical latitudes, the frozen state is almost never encountered. The proportion between liquid and frozen hydrometeors also affects the power returned, causing difficulties in accurate estimation of precipitation rates.

Radar Equation

Reflectivity values are obtained from radar measurements of the backscattered power for a particular DSD. If the object is small in relation to the wavelength of the impinging radiation, the Rayleigh approximation is used. Under this assumption, Rayleigh backscattering of radiation of wavelength λ from raindrops of diameter D is valid when $D < \lambda/16$. The WSR-88D radar operates at a frequency equivalent to a wavelength of 10 cm (also referred to as S-band). Raindrops rarely exceed 4 or 5 mm before they break up into smaller drops. Backscattered radiation by raindrops measured with a 10-cm-wavelength radar appropriately uses Rayleigh scattering to estimate the rainfall rate, since the permissible diameter $\lambda/16 = 100/16 = 6.25$ mm. Objects in the atmosphere larger than 4 or 5 mm differ in scattering properties and would be described by the Mie rather than the Rayleigh backscattering property (AMS, 2000).

In radar meteorology, the Probert–Jones equation is also known as the radar equation (Doviak and Zrnica, 1993). The Probert–Jones equation for meteorological radar relates received radar-measured power, P_r , to characteristics of the radar and to characteristics of the precipitation targets:

$$P_r = \frac{CLZ}{r^2}, \quad (11-1)$$

where P_r denotes radar-measured power; C is a constant that depends on radar design parameters such as power transmission, beam width, wavelength, and antenna size; L represents attenuation losses; Z is the radar reflectivity of the target; and r is the range to the target. Reflectivity, Z , is expressed in units of mm^6/m^3 , or the sixth power of the diameter of raindrops in mm per cubic meter.

Because the returned power decreases according to the square of the distance, the WSR-88D radar-signal processing must compensate for this variation over a range of 2 to 230 km. The range-correction algorithm is also designed to compensate for variations associated with incomplete beam filling and the height of the beam in the atmosphere. The assumption underlying range correction is that precipitation is detected, but underestimated, because of the large sample volume and the elevation in the atmosphere. The Probert-Jones equation combined with the range-correction algorithm is used to calculate the reflectivity of the rainfall droplets from the radar-measured power of the return signal.

Reflectivity–Rainfall Relationship

What is rain? While we all have ideas about rain from experience, to understand how radar is used to estimate rainfall we must examine rainfall from the radar point of view. To determine the rainfall rate from the reflectivity, we establish a relationship between DSD and rainfall rate. The resulting relationship of reflectivity to rainfall rate is known as a *Z-R* relationship and is the primary means for estimating rainfall with radar.

The radar return power from a raindrop is proportional to the sixth power of the diameter of the raindrop. If you separate the raindrops from one cubic meter of air into n size categories, the sum of D^6 over all the particles and size categories is equal to the radar reflectivity factor Z , defined as

$$Z = \frac{1}{v} \sum_{i=1}^n N_i D_i^6, \quad (11-2)$$

where n is the number of drop-size categories in the sample volume; N_i is the number of drops in each size category, i , per unit volume, v ; and D_i is the diameter (mm) of size category i . While units of Z in Equation (11-2) are mm^6/m^3 , they are usually transformed using logarithms and stated in terms of decibels or dBZ.

In order to complete the summation in Equation (11-2), one must know the number of drops in each drop-size category per unit volume. Probabilistic approaches are employed. Marshall and Palmer (1948) proposed a DSD based on measurements using an exponential probability distribution, which is one form of the gamma distribution known as the Marshall–Palmer DSD. The number of raindrops, N , of size D (mm) has units of drops per cubic meter per mm ($\text{mm}^{-1} \text{m}^{-3}$). When integrated over the range of drop sizes (mm), the units of N become drops per cubic meter (m^{-3}). The number of drops, N , per cubic meter is

$$N = N_0 e^{-\Lambda D}, \quad (11-3)$$

where $N_0 = 8000 \text{ (mm}^{-1} \text{m}^{-3}\text{)}$ is the number of drops at diameter zero (the number of drops does not vanish as the diameter approaches zero,

but converges to N_0), and $\Lambda = 3.67/D_0$ is related to the median drop size with units of mm^{-1} . The parameter Λ is also related to the rainfall rate R , so that the rainfall changes with DSD. The DSD is thus the common linkage between the reflectivity Z and the rainfall rate R . The Marshall–Palmer DSD parameters are taken as $N_0 = 8000 \text{ (mm}^{-1} \text{ m}^{-3}\text{)}$ and $\Lambda = 3.67/D_0 = 4.1R^{-0.21}$. Figure 11–4 shows the distribution of drop sizes for three different rainfall rates: 1, 10, and 100 mm/hr, illustrating the dependence of rain rate on the DSD.

Substituting Equation (11–3) into Equation (11–2) and integrating over all drop sizes in the DSD yields

$$Z = \int_0^{\infty} N_0 e^{-\Lambda D} D^6 dD = N_0 \frac{\Gamma(7)}{\Lambda^7}, \quad (11-4)$$

where $\Gamma(7)$ is the gamma function, which in this case is factorial 6, or $6!$, and other terms are as defined previously. Equation (11–4) tends to overestimate the reflectivity if it is integrated over the range $0 \rightarrow \infty$. More realistically, the upper integration limit should be taken as D_{max} , the maximum drop size (generally less than 3–4 mm diameter), rather than infinity. Substituting the Marshall–Palmer DSD parameters ($\Lambda = 3.67/D_0$) into Equation (11–4) results in a more computationally convenient form for the reflectivity Z :

$$Z = 0.080 N_0 D_0^7, \quad (11-5)$$

where D_0 is the median drop size. Thus, reflectivity is related to the two parameters of the drop-size distribution.

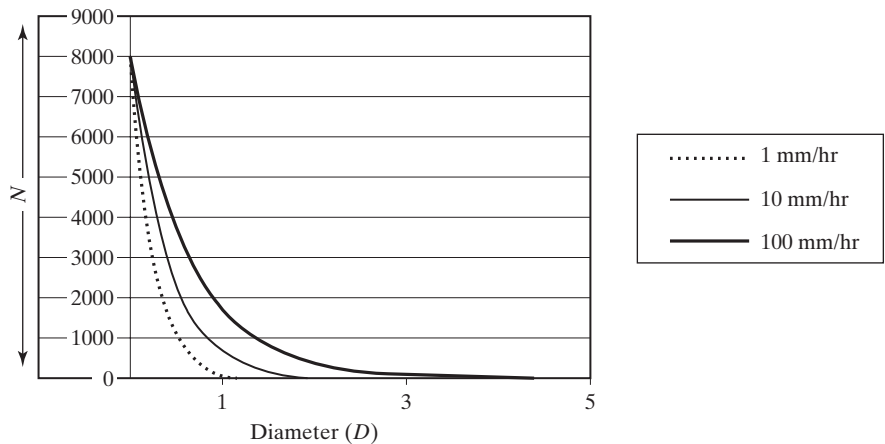


Figure 11–4

Marshall–Palmer DSD for rainfall rates 1, 10, and 100 mm/hr.

ESTIMATING REFLECTIVITY BASED ON DSD PARAMETERS**EXAMPLE 11-1**

Rearranging Equation (11-4) in terms of the media drop size, the following computational form results:

$$Z = N_0 \frac{6!}{(3.67/D_0)^7} = 0.080 N_0 D_0^7.$$

For example, given the number of drops, $N_0 = 8000$ raindrops ($\text{mm}^{-1} \text{m}^{-3}$), and the median drop size, $D_0 = 2.42$ mm, the corresponding reflectivity factor would be

$$Z = (0.080)(8000)(2.42)^7 = 311,091 \text{ (mm}^6 \text{m}^{-3}\text{)}.$$

Because the reflectivity factor may cross several orders of magnitude, it is more convenient to work with decibels of reflectivity. By convention, a decibel is $10 \log_{10}$ of some number. A decibel of reflectivity is termed a dBZ; thus,

$$10 \log(Z) = 10 \log(311,091) = 54.9, \text{ or } \sim 55 \text{ dBZ}.$$

When working with radar data, the reflectivity is almost always reported in dBZ. As will be seen, reflectivity of 55 dBZ corresponds to very intense precipitation. Because the reflectivity factor is reported in decibels, small changes in this logarithmic quantity represent large changes in the rainfall rate.

The distribution parameters of the DSD are used to compute not only reflectivity but also rainfall rate, R . The rainfall associated with R is not actually measured directly by the radar, but rather is inferred from the fall velocity of the raindrops based on the diameter estimated from reflectivity. Atlas and Ulbrich (1977) reported an empirical formula for the fall velocity, $V_D = 386.6 D^{0.67}$ (m/s), for rainfall drop sizes, D , in meters. A median drop size, $D_0 = 2.42$ mm or 0.00242 m, will fall at approximately 6.8 m/s in calm air. Downdrafts (updrafts) during a thunderstorm cause a localized increase (decrease) in the rainfall rate because V_D is increased (decreased) from the fall velocity. The localized effects of updrafts and downdrafts, among others, cause rainfall rates estimated aloft by radar to differ from those measured on the ground at point locations using gages. Variations in fall speed and departures from the assumed DSD increase the systematic and random errors associated with radar rainfall.

Using techniques similar to those used to calculate the reflectivity from the DSD, the rainfall rate may be calculated from the DSD. Rainfall rate, R , expressed as depth per unit time, is related to the DSD and the fall velocity by

$$R = \int_0^{\infty} (\text{number} \times \text{velocity} \times \text{volume}) dD = \frac{N_0 D_0^{14/3}}{4026} \text{ (mm/hr)}, \quad (11-6)$$

where N_0 and D_0 are as defined previously. As with Equation 11–5, the rainfall rate is also related to the two parameter drop-size distribution. Derivation of Equation (11–6) is left to the reader as an exercise.

Now that we have Z -DSD (Eq. 11–4) and R -DSD (Eq. 11–6) relationships, we can combine the two equations to form a single Z - R relationship. Example 11–2 illustrates two ways to arrive at a Z - R relationship, algebraically and empirically. Battan (1973) lists as many as 69 different Z - R relationships. Efforts to classify the type of precipitation mechanism (e.g., thunderstorms, stratiform, or tropical) and then use an appropriate Z - R relationship may reduce the number of such relationships applicable to a particular storm. However, due to uncertainty in the DSD and resulting reflectivity, adjustment with rain gages is still necessary. Other factors that cause uncertainty in this relationship are differences in transmitted and received radar power, which also may give rise to the large number of Z - R relationships (Doviak and Zrnic, 1993).

Marshall and Palmer (1948) measured Z -DSD and R -DSD relationships for drop sizes between 1.0 and 3.5 mm diameter. The combination of those measured relationships resulted in the empirically derived Marshall–Palmer Z - R relationship,

$$Z = 200R^{1.6}. \quad (11-7)$$

Several other Z - R relationships have been derived and will be discussed in subsequent sections. Because the diameter is raised to the sixth power, changes in drop size and numbers of these raindrops can have dramatic effects on the estimated rainfall rate. In practical applications, the Z - R relationship most appropriate for the type of storm (tropical, convective, or stratiform) is applied to reflectivity and rainfall rates derived. Example 11–2 demonstrates the basis of the Z - R relationship on assumed DSD parameters.

EXAMPLE 11–2

RELATIONSHIP BETWEEN RAINFALL RATE R AND REFLECTIVITY Z FOR ASSUMED DSD PARAMETERS

Taking Equation (11–5) for reflectivity Z and Equation (11–6) for rainfall rate R (mm/hr), we can algebraically derive a Z - R relationship, as follows:

$$Z = 0.080N_0D_0^7,$$

$$R = \frac{N_0D_0^{14/3}}{4026}.$$

Solving for D_0^7 , we get

$$R^{3/2} = \left[\frac{N_0D_0^{14/3}}{4026} \right]^{3/2} = \frac{N_0^{3/2}D_0^7}{4026^{3/2}},$$

$$D_0^7 = \frac{4026^{3/2}R^{3/2}}{N_0^{3/2}}.$$

Substituting D_0^7 into the Z equation and using $N_0 = 8000 \text{ (mm}^{-1}\text{m}^{-3}\text{)}$, we get

$$Z = 228R^{1.5}(\text{mm}^6 \text{ m}^{-3}).$$

We will assume that we have measured Z and R for a range of D_0 values, and we did not have an algebraic relationship. First, we will estimate Z and R for the same D_0 values from our measured Z -DSD and R -DSD relationships (actually, these numbers are calculated from the previous equations for a range of drop sizes, assuming $N_0 = 8000$).

D_0 (mm)	Z (mm^6m^{-3})	R (mm/hr)
1	2	3
1.00	640	2
1.25	3052	6
1.50	10,935	13
1.75	32,170	27
2.00	81,920	50
2.25	186,835	87
2.50	390,625	143

Plotting column 2 vs. column 3 in the table results in a Z - R graph that can be used to fit an equation of the form $Z = aR^b$, where a and b are determined by least-squares analysis or with Excel's trendline function.

In Figure 11-5, we can compare the two relationships and see the difference in the Z - R relationship caused by variations in the DSD and in the R -DSD and Z -DSD relationships. The empirically derived Marshall-Palmer relationship of $Z = 200R^{1.6}$ uses a drop-size range of 1.0 to 3.5 mm,

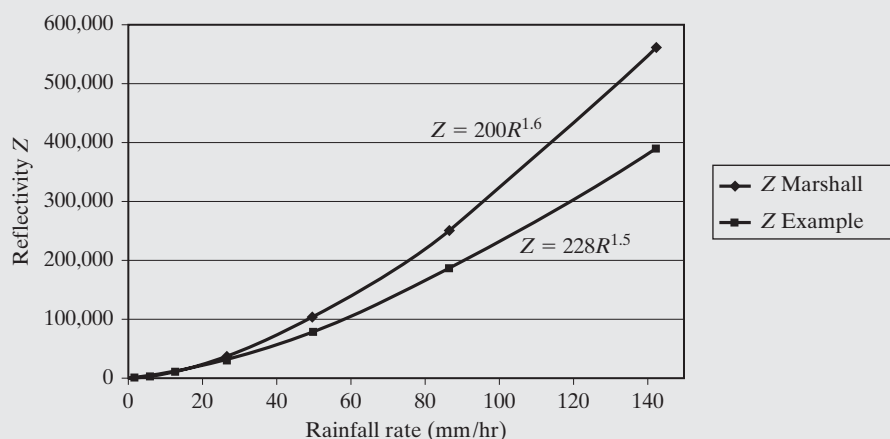


Figure 11-5

Comparison of analytic and empirically derived Z - R relationships.

while the analytically derived relationship $Z = 228R^{1.5}$ uses all drop sizes from $0 \rightarrow \infty$, yielding higher rainfall rates. Practically, D should be capped at some maximum drop size when integrating these relationships. This example demonstrates that the Z - R relationship depends heavily on the distribution of drop sizes.

Because the assumed DSD usually differs from the actual storm, the resulting radar rainfall estimates are biased upward or downward, which is a systematic error. Atlas and Chmela (1957) report a number of radar-estimated rainfall rates for the same type of precipitation event varying by as much as 300%. Doviak and Zrnic (1993) suggest that part of the wide variation in Z - R relationships may in fact be due to calibration of the radar that affects the transmitted power. It is difficult to calibrate radars to within ± 1 dB of power. Because two radars may have different power characteristics, they are often referred to as being comparatively *hot* or *cold*. Rainfall estimates, made at the same location but measured from two radars, may differ due to the power differences transmitted and received and the elevation of the target in the atmosphere as viewed by the radars.

To overcome systematic errors (bias) inherent in the Z - R relationship, radar correction through adjustment with rain gages can be performed in real time, or in postanalysis using archival radar and gage data. This procedure usually consists of comparing accumulations between radar and gage. The ratio of the radar to gage measurement is termed a **bias correction factor**. When the bias correction is computed as a mean field bias, the correction procedure consists of comparing many radar/gage pairs of accumulations and then averaging to obtain a mean correction over some geographic area or window. If the radar is underestimating by 20%, the rainfall fields are increased by 20% to compensate for the bias. Adjustment of the radar to agree with rain gage accumulations, using the method of Wilson and Brandes (1979), is the most common technique for correcting radar rainfall estimates. In effect, such calibration is an adjustment of the multiplicative constant in the Z - R relationship. For example, during a major rainfall event in southeast Texas in October 1994, the WSR-88D radar at Houston-Galveston (KHGX) underestimated the rainfall by as much as 50%. In a reanalysis of the radar reflectivity, Vieux and Bedient (1998) found that the tropical Z - R ($Z = 250R^{1.2}$) relationship better characterized the October 1994 storm event than the Z - R relationship used by the NWS at the time of the event. Using the tropical Z - R to compute daily accumulations, the mean field bias for this event ranged from 6% underestimation on October 17 to 15% overestimation on October 18. From this study, the WSR-88D radar was found to be a viable source of rainfall information for flood warning and hydrologic prediction, provided that an appropriate Z - R relationship

is used. While the use of an appropriate Z - R relationship will reduce the difference between rain gage and radar rainfall accumulations, adjustment of the Z - R relationship through comparison with rain gage accumulations is usually required before the radar-derived rainfall can be used for hydrologic purposes.

When gage-adjusted radar is used as rainfall input to lumped or distributed hydrologic models, the hydrograph rising limb and peak flow are typically more accurate than the hydrographs produced from rain gage data alone. The fundamental advantage of radar over rain gage networks for rainfall estimation is the density of measurement provided by the radar. Measurement strategies are dictated by the radar system characteristics and intended use, whether for severe storm and tornado detection or precipitation estimates. The following section describes the high-resolution precipitation products available from the WSR-88D system.

The WSR-88D radar is called a volume-scanning radar, because as the antenna rotates, it covers the surrounding atmosphere with successive tilts, beginning at 0.5-degree angles and incrementing approximately by 1 degree with each revolution. These measurements extend over the entire volume of the atmosphere, out to 460 km in range for reflectivity and 230 km for precipitation. Depending on the volume-scanning pattern (VCP), the rate at which each volume scan is completed varies from 5 or 6 minutes during precipitation to longer durations during *clear air* mode. Rates of data acquisition are 12, 10, and 6 volume scans per hour for VCP 11, VCP 21, and VCP 32, respectively. These variable rates have important consequences on the reflectivity and derived precipitation products. From a hydrologic viewpoint, the VCP determines the time resolution of rainfall intensities and depth estimates. During a storm event, a NEXRAD radar operator may switch from VCP 11 to VCP 21 and back several times, causing variable time-step precipitation estimates or hyetographs. New volume scanning strategies have been put into practice by the NWS to measure quickly evolving severe storms more rapidly in less than 2–3 minutes. To use derived estimates of rainfall, interpolation to constant-time interval is often necessary for use with hydrologic models such as SWMM or HEC-HMS (HEC-1). Radar system characteristics of the WSR-88D radar are described next.

11.3 NEXRAD (WSR-88D) RADAR SYSTEM

Precipitation Processing System Characteristics

The WSR-88D precipitation processing system (PPS) generates products useful for weather forecasting. There are several precipitation products with varying degrees of spatiotemporal resolution and data precision. These products, as well as components comprising the NWS Precipitation Processing System, are described in Fulton et al. (1998) and in the sections that follow.

The Level II base data generated by the NEXRAD system are composed of reflectivity, Doppler velocity, and spectrum width. Reflectivity is of interest to hydrologists because the reflectivity data, reported in decibels of reflectance, dBZ, may be transformed into rainfall rates. Z - R relationships, as demonstrated, depend on a number of factors, including the DSD, making it necessary to calibrate the resulting rainfall rates with rain gages. During the stages of precipitation processing by the system, NWS algorithms are designed to remove spurious reflectivity caused by sidelobe contamination, ground clutter, and anomalous propagation of the beam; reflectivity is converted to rainfall rate using a Z - R relationship; and rain rates are accumulated over time and adjusted based on rain gage reports. Provisions have been made for calibrating to rain gages located under the radar umbrella, although this has not been fully realized operationally by the NWS. Processing rainfall information from Level II data independently from the PPS has the advantage of yielding the finest space/time resolution data available, though still in polar coordinates of 1 km in range by 1 degree in azimuth. Dissemination of “super resolution” Level II data is planned for the NEXRAD system, which will be at 0.25-km range resolution and one-half degree azimuth.

Rainfall rates are estimated from radar using a selected Z - R relationship. In postanalysis of storm events, if the reflectivity data are available, the analyst should apply an appropriate Z - R relationship. Operationally, the radar operator selects the most appropriate Z - R relationship according to the season or type of event producing the precipitation. The standard Z - R relationship used with the initial installation of all WSR-88D radars in the United States was $Z = 300R^{1.4}$. Since deployment, the NWS has adopted several Z - R relationships that are applicable to convective, tropical, winter-time stratiform storms, among others. The tropical Z - R relationship, $Z = 250R^{1.2}$, is more representative of warm rain processes that occur in areas near the tropics, e.g., coastal areas of Texas to Florida. The tropical Z - R is more appropriate for tropical storms or for storms from clouds that have temperatures $>32^{\circ}\text{F}$ producing rain from coalescence of droplets (AMS, 2000). Precipitation products currently generated by the NEXRAD radar product generator and single radar images obtained from the NWS via the web may not be gage adjusted.

In a computationally convenient form, the conversion from reflectivity (dBZ) to rainfall rate (R in mm/hr) can be made using the following equations for the tropical Z - R and standard Z - R relationships:

Tropical Z - R relationship $Z = 250R^{1.2}$ can be rearranged to

$$R = \left(\frac{10^{\text{dBZ}/10}}{250} \right)^{(1/1.2)} = \frac{10^{\text{dBZ}/12}}{99.6}. \quad (11-8)$$

Standard Z - R relationship $Z = 300R^{1.4}$ can be rearranged to

$$R = \left(\frac{10^{\text{dBZ}/10}}{300} \right)^{(1/1.4)} = \frac{10^{\text{dBZ}/14}}{58.8}. \quad (11-9)$$

Rainfall rates produced from the two Z - R relationships [Eqs. (11–8) and (11–9)] are compared in Table 11–1 for a typical range of reflectivity. The reflectivity recorded for each sector of the volume scan is in dBZ of reflectance, or $10 \cdot \log_{10}(Z)$. The logarithmic scale is important to keep in mind when comparing reflectivity. Seemingly small increases in detected reflectivity on a logarithmic scale represent large increases in rainfall rate. An increase of just 5 dBZ (e.g., from 35 to 40 dBZ) results in more than a threefold increase in Z , which translates into 21.63 mm/hr compared with 8.29 mm/hr in the tropical Z - R relation. For this reason, the precision of the radar reflectivity data recorded by the precipitation processing system becomes important. More levels of precision, referred to as quantizing, correspond to more precise rainfall estimates.

The large differences between a tropical and standard Z - R relationship are evident in the last two columns of Table 11–1. If a reflectivity of 45 dBZ were detected, the tropical relation would estimate the rainfall rate to be 56.46 mm/hr, compared with 27.86 mm/hr for the standard. In practice, the appropriate Z - R relationship should be selected based on the physical meteorological character of the storm producing the precipitation. The underlying physics of the rainfall-producing process governs the DSD and can only be guessed at by the radar operator. A high reflectivity cap is also applied to reduce the influence of water-coated hydrometeors such as hail. This cap is an adaptable parameter at the radar installation but is usually set to 53 dBZ, which corresponds to 103.8 mm/hr, using the standard convective Z - R relation.

A brief description of the WSR-88D radar data acquisition and processing stream is useful for understanding the data source, quality, and type of precipitation data available for use in hydrologic modeling. Depending on where you receive data from the system, data of varying precision, resolution, and format will result. Reflectivity and precipitation products displayed on the web by the NWS are in an image format and have lower data precision, typically with only 16 data levels depicted by the color scale. Furthermore, web-based products are likely not gage adjusted or in a georeferenced coordinate system. Crum and Alberty

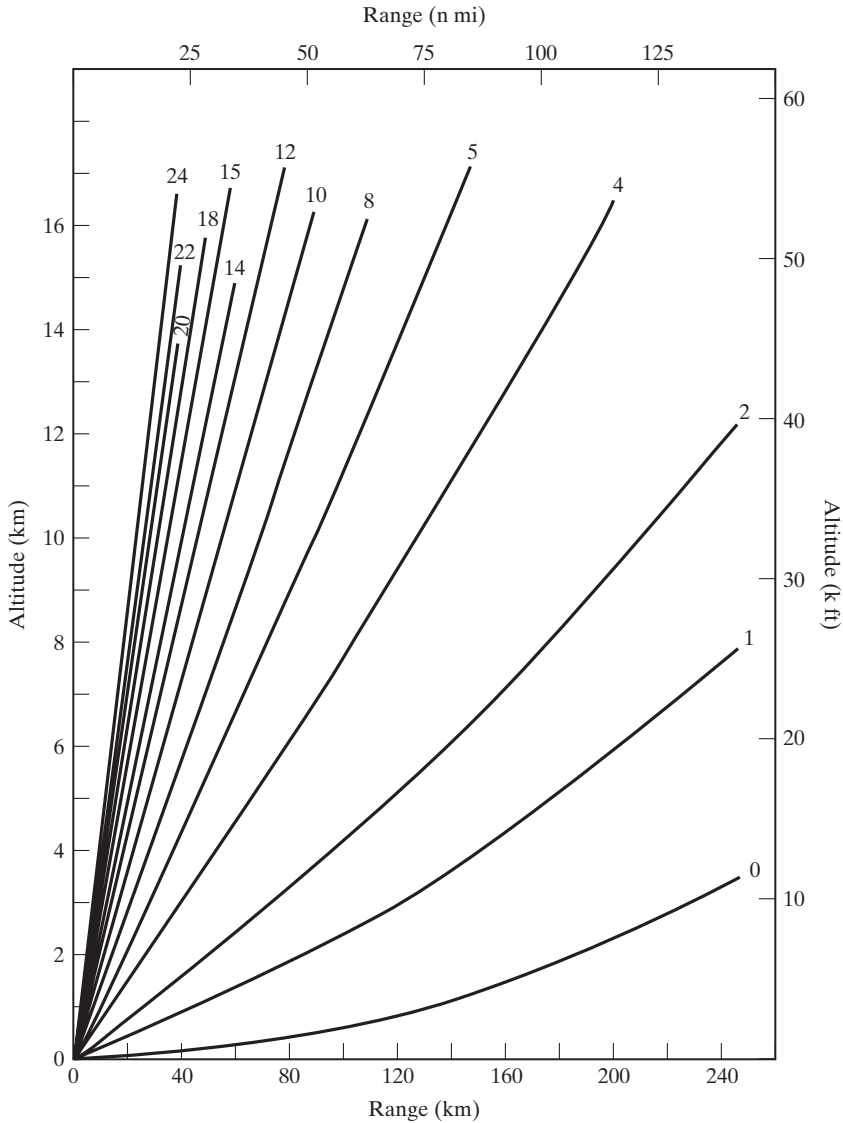
Table 11–1 Comparison of Z - R Relationships for a Range of Reflectivity

Z	dBZ	Tropical (in./hr)	Standard (in./hr)
100	20	0.02	0.02
316	25	0.05	0.04
1000	30	0.12	0.09
3162	35	0.32	0.21
10,000	40	0.84	0.47
31,623	45	2.19	1.08
100,000	50	5.71	2.46

(1993) describe the initial WSR-88D system, which is an evolving system. More recent description of the radar system, theory, operation, and meteorological data products and processing may be found in FCM-H11A (2006), FCM-H11B (2005), FCM-H11C (2006), and FCM-H11D (2006). Radar data and derivative products can be accessed by users outside of the WSR-88D agencies. Real-time access to products and Level II data is provided through two networks managed by the NWS: (1) Radar Product Central Collection Dissemination Service (RPCCDS) and (2) Level II data obtained through the NWS Level II Data Collection and Distribution Network. Archival Level II data produced by the system are also available from the National Climatic Data Center. Value-added precipitation products that have been quality controlled and enhanced for accuracy through gage correction are available from private-sector vendors in hydrologic model and GIS formats.

The major components of a WSR-88D radar unit are: (1) Radar Data Acquisition (RDA), (2) Radar Product Generator (RPG), (3) Associated Principal User Display Systems, and (4) Communications. The RDA provides the detection and measurement of meteorological and hydrological phenomena. The RDA functional area consists of the hardware, firmware, and software required to control the antenna; transmit and receive radio-frequency pulses; perform signal processing, ground clutter suppression, and range unfolding; and distribute base data in the form of three base moments (reflectivity, mean radial velocity, and velocity spectrum width) to the RPG. The RDA is composed of the antenna, pedestal, radome, facilities, transmitter, receiver, signal processor, user interfaces, and communications.

The base data is collected at various tilt elevation angles of the radar. The beam height and range are plotted in Figure 11–6 for the tilt elevations measured by the radar during the volume scanning of the atmosphere. Data from the RDA are referred to as Level II and consist of unprocessed (raw) reflectivity, Doppler wind velocity, and spectrum width. For hydrologic purposes, reflectivity data are in general the only data collected that are of interest. The next phase in the processing stream is the RPG, which applies computer algorithms to produce the four precipitation processing system (PPS) products: 1-hr accumulation, 3-hr accumulation, storm total accumulation, and hourly digital precipitation. Data from the RPG are referred to as Level III. During the RPG processing, the data are aggregated in time and space, depending on the product generated. The precipitation products generally available to the public are aggregated at various resolutions and time intervals. The Stage 1 product is generated at individual radar installations and generally has not been adjusted to rain gage data. After the data stream leaves the individual radar installation, gage bias corrections are made at Stage 2, and multiple radars are mosaiced together to form what is called a Stage 3 product over river forecast areas, such as the Arkansas–Red River Basin, among others. At Stage 4, a national mosaic is formed as in

**Figure 11-6**

Radar beam altitude and range for tilt angles above horizontal.

Figure 11-7. The national radar mosaic in Figure 11-7 is produced from Stage 3 mosaics and shows radar reflectivity for the entire United States during a widespread winter storm event occurring in the central states of Kansas, Oklahoma, and Texas at 20:08 UTC (universal coordinated time) on January 20, 2007. The reflectivity data are produced by combining many radars in different local time zones to form the mosaic.

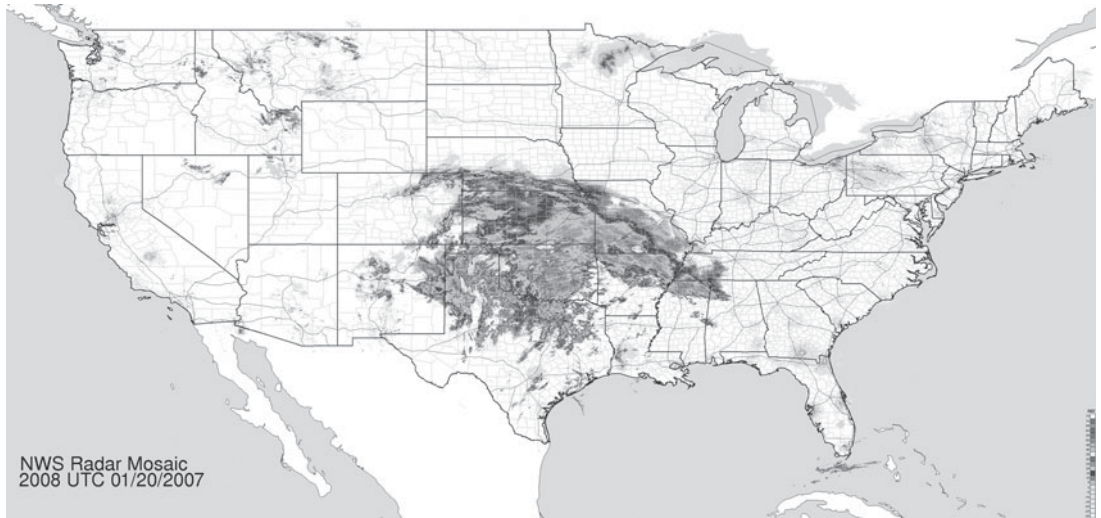


Figure 11-7

National radar mosaic showing the latest reflectivity for the entire United States valid at 20:08 UTC on January 20, 2007.

11.4 GAGE ADJUSTMENT OF RADAR

With adequate maintenance, calibration, and data archiving, rain gage networks provide important point measurements of rainfall, which are useful for rainfall estimation. Any measurement is subject to measurement errors. Rain gage measurement accuracy and precision depend on the type of gage, periodic maintenance, rain shadowing by adjacent obstructions, clogging, malfunctioning telemetry, and wind- and evaporation-induced losses. After rain gage data have been checked for missing data and systematic and random errors, they are used in conjunction with radar to estimate rainfall.

The process for combining radar and gage estimates relies on computing a systematic error, or bias correction factor, and applying the correction to the unadjusted radar rainfall accumulations. Computing a single bias correction factor that is spatially constant is called a mean field bias correction. Allowing the correction factor to vary in space is termed a local bias correction. Seo et al. (1999) and Seo and Breidenbach (2002) describe the mean field and local bias approach to gage correction of radar. After bias correction, random errors related to scale differences between gage orifice and radar sample volume and updraft/downdraft effects will remain. After adjusting for systematic error, the radar estimate is consistent, though not necessarily in precise agreement with rainfall measured at a point location represented by the gage.

Mean Field Bias

Wilson and Brandes (1979) described removal of systematic errors in radar estimates using rain gage accumulations. Bias in radar terms means that the radar is overestimating or underestimating, compared with gage accumulations. At any one gage, the radar may be overestimating or underestimating. However, the mean over many gages provides a statistical basis for removing systematic bias. Pairs composed of gage (G_i) and radar (R_i) rainfall accumulations may be compared on an hourly, daily, or storm-total basis. Two methods are commonly used for correcting radar estimates: (1) ratio of the means (RM) of the gage–radar pairs and (2) mean of the ratios. Correction of the radar bias is accomplished by applying the multiplicative factor F_{RM} , which removes the mean field bias (MFB), defined as

$$F_{RM} = \frac{\sum_{i=1}^n G_i}{\sum_{i=1}^n R_i}, \quad (11-10)$$

where G_i and R_i are the i th gage–radar pairs of accumulations and n is the number of pairs. When computed for a storm event total, G and R represent total accumulations for the event. This factor is recognized as the ratio of the means of G and R , respectively, or $F_{RM} = \bar{G}/\bar{R}$, whereas the factor F_{MR} is the mean ratio or $\bar{G}/\bar{R}$. Relative dispersion gives an estimate of the cluster about the mean. It is computed as the standard deviation σ of the G/R pairs, normalized by the multiplicative factor F_{RM} . The relative dispersion, RD , is the standard deviation of the gage–radar pairs after bias correction and is defined as

$$RD = \sigma\left(\frac{G_i}{R_i}\right), \quad (11-11)$$

The normalizing factor F_{RM} or F_{MR} will have a value of 1 when there is perfect agreement between gage and radar, i.e., no bias.

Bias is removed from the radar by multiplying each cell value by the multiplicative factor, F , computed by either formula, using $\bar{G}/\bar{R}$ or $\bar{G}/\bar{R}$. Once the systematic bias is removed, random errors remain. The accurate determination of random errors is accomplished by computing the average difference between the gage and radar accumulations after the bias removal. The average difference $\bar{D}$ is defined as

$$\bar{D} = \frac{100\%}{n} \sum_{i=1}^n \left| \frac{G_i - F_{RM} * R_i}{G_i} \right|, \quad (11-12)$$

where F_{RM} is the multiplicative factor defined above. The average difference, $\bar{D}$, is a measure of the uncertainty of the radar estimates compared to rain gage accumulations. This statistic can be considered as a measure of random error after the systematic bias is removed, using the multiplicative factor F_{RM} .

EXAMPLE 11-3**RADAR ADJUSTMENT USING RAIN GAGES**

The radar rainfall event analysis period was from 7/12/01 1:00 CDT to 7/12/01 10:00 CDT (7/12/01 6:00 UTC to 7/12/01 15:00 UTC). Level II NEXRAD data from the primary radar, KEAX, were used to process the event. No gage outliers were detected during this event. The convective Z - R relationship, $Z = 300R^{1.4}$, was used to convert radar reflectivity to rainfall rates. Table E11-3-1 summarizes the results for each gage-radar pair used for radar adjustment, where G_i is the gage estimate, R_i is the nonadjusted radar estimate, R_i^* is the adjusted radar estimate, Diff* (in.) is the difference in inches between the gage and adjusted radar estimate, and Diff* (%) is the percent difference between the gage and adjusted radar estimate. The Bias column is the ratio of gage/radar. The distribution of this ratio by spatial interpolation is multiplied times the raw radar accumulation, R_i , to obtain the corrected radar accumulation, R_i^* . Note that the gage-adjusted radar at any given location is the result of distance-weighted correction. For this reason, the individual radar accumulations are not a simple product of the local bias and the unadjusted radar accumulation, but rather a combination produced from surrounding gages. After gage adjustment, the percent difference, Diff*(%), ranges from -11.8 to 10%, whereas, in the last column, the Diff* (in.) ranges from -0.09 to .014 in. The Bias ranges from 1.60 to 0.82, indicating a range of correction factors that increase or decrease the radar, depending on location. The variability of the Bias may be due to a variety of factors already discussed, including difference in DSD and updrafts/downdrafts. The mean field bias correction factor is computed by the mean gage and mean radar accumulation as

$$F_{RM} = \frac{G}{R} = \frac{1.01}{0.88} = 1.14,$$

which represents a mean field bias correction factor because it is computed as a mean of the radar and gage accumulations.

Table E11-3-1 Summary of Individual Radar-Gage Storm Total Accumulations

Gage	G_i	R_i	R_i^*	Bias	Diff* (in.)	Diff* (%)
RG 02	0.73	0.89	0.82	1.60	-0.09	-11.8
RG 04	1.11	0.93	1.20	0.82	-0.09	-8.0
RG 06	0.79	0.72	0.78	1.02	0.01	1.1
RG 03	1.03	1.01	1.01	1.19	0.02	2.4
RG 01	1.39	0.87	1.25	1.09	0.14	10.0
Mean =	1.01	0.884	1.012	1.144	-0.002	-1.26

When the correction factor, F_{RM} , defined in Equation 11-10, is applied to the radar accumulations, the radar will be increased by 14%, and thus removing the bias. Applying the spatial constant correction factor is termed a mean field bias correction. If the correction variable is allowed to vary in space, then a local bias correction is achieved. In this example, the correction factor is applied based on a storm total. Alternatively, the mean field or local bias correction may be updated during the storm evolution in time. In real-time applications, the correction must be computed as a running average in time to account for the variability of correction factors in time. Note that the mean bias factor is $F_{RM} = 1.14$, but varies in space (across the gage network) from a low of 0.82 to a high of 1.60.

Figure E11-3-1(a) is the unadjusted gage-radar scatter plot, and Figure E11-3-1(b) presents the scatter plot of the adjusted radar and gage totals. The slope of the trend line in the unadjusted radar of 0.84 indicates that the radar is underestimating by about 16%. If greater than 1.0, the bias correction factor indicates underestimation by the radar, which in this case is about 14%. After gage adjustment, the bias is nearly gone, as shown by the slope of the best-fit line of 0.988, or about one to one.

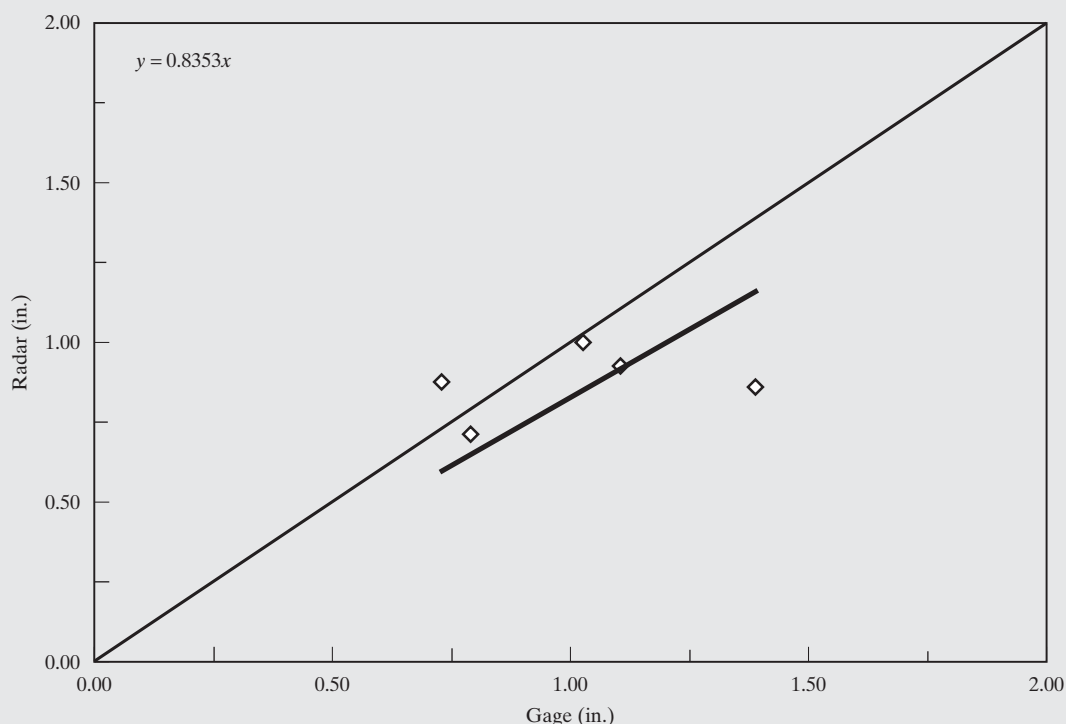


Figure E11-3-1(a)

Unadjusted scatter plot of calibrated radar and gage storm totals.

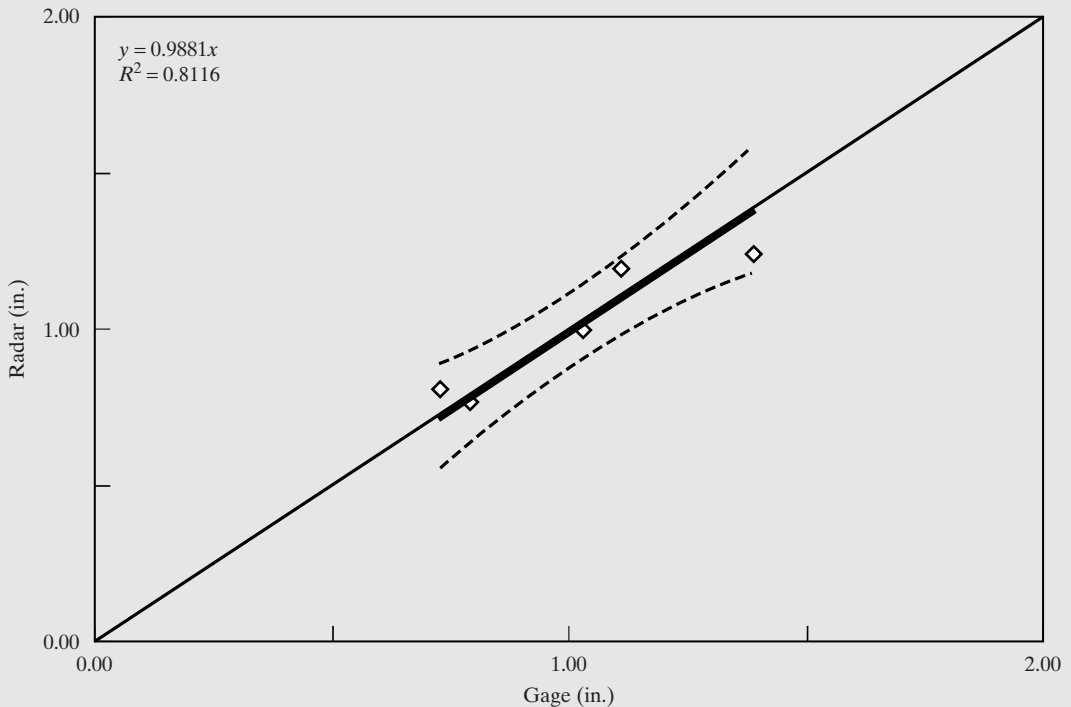


Figure E11-3-1(b)

Gage-adjusted scatter plot of calibrated radar and gage storm totals.

After producing gage-adjusted rainfall maps, the data may be aggregated to watershed areas in the form basin averages. The time history can be reconstructed from the radar accumulations, which are generally collected every 5 or 6 minutes depending on the scanning strategy of the radar during operations. The time distribution of gage rainfall and adjusted/unadjusted radar may be plotted along with gage accumulations, as shown in Figure E11-3-2. The adjusted radar rainfall cumulative hyetograph will not agree precisely with the gage because the plot shown is for a local bias adjustment, where the correction is the result of the local and surrounding gages. The adjusted radar accumulation agrees more closely with the gage than does the unadjusted radar. Other differences may be due to the scale difference between the radar sample volume on the order of one or more square kilometers and the rain gage on the order of 8 inches for a standard NWS gage.

The comparison of temporal evolution of the radar and gage hyetographs provides useful information for quality control and evaluation of gage performance during the event. Even after the radar is bias corrected, perfect agreement between the gage and radar accumulations is not

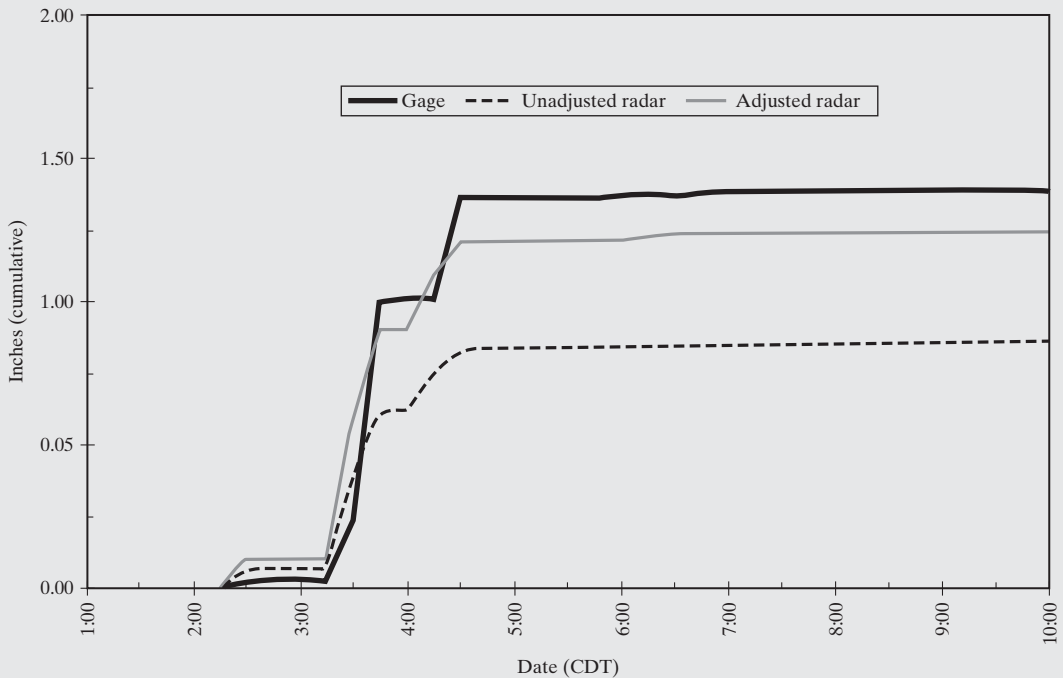


Figure E11-3-2

Hyetographs of gage and radar (adjusted and unadjusted).

expected due to local effects of updrafts, downdrafts, and scale effects between radar sample volume and the size of the gage orifice. After the radar is adjusted by the procedure described in this example, radar rainfall maps are generated as a time series for hydrologic model input.

From radar, hyetographs aggregated for each subbasin or grid may be used for input to a hydrologic model. Besides file format manipulation, the link between radar data and hydrologic models requires spatial aggregation from one grid to another or to subbasin areas. Aggregation from polar coordinates to basins or rectangular grids is usually accomplished with GIS or special-purpose tools that can handle spatial data in geographic projections. When rainfall data are aggregated from polar grids to subbasin definitions, they may be used as input to hydrologic models such as HEC-HMS for surface runoff, or such models as InfoWorks, Mouse, or SWMM for sewer model applications. Radar rainfall that is gage adjusted is used for hydraulic modeling in the design and rehabilitation in both combined and separate sanitary sewers. Quantifying the rainfall distribution in space and time for

11.5 HYDROLOGIC APPLICATIONS

each catchment in the model simplifies the model calibration process, because more representative rainfall makes the modeled results more reliable (Burgess et al., 2003; Wride et al., 2003).

Gage-adjusted rainfall on August 28, 2006, over Hamilton County and Greater Cincinnati, Ohio, is shown as a storm total map in Figure 11–8. The rainfall is shown in polar coordinates along with an outline of sewer catchments and rain gage locations. The rainfall totals in Figure 11–8 as polar sample volumes are aggregated to subbasin definitions in Figure 11–9. The subbasin aggregation produces hyetographs that are representative of rainfall over each model subbasin or catchment for input to a hydraulic model, SWMM. Radar precipitation measurement captures more variability than would be achieved from using adjacent rain gages. The rain gage network density over Hamilton County is variable but averages one gage per 33 km² based on a bounding box. The rainfall variability, indicated by the gray shades in Figures 11–8 and 11–9, is considerably greater than could be captured by the rain gage network. Rain gages inside and outside of the modeled

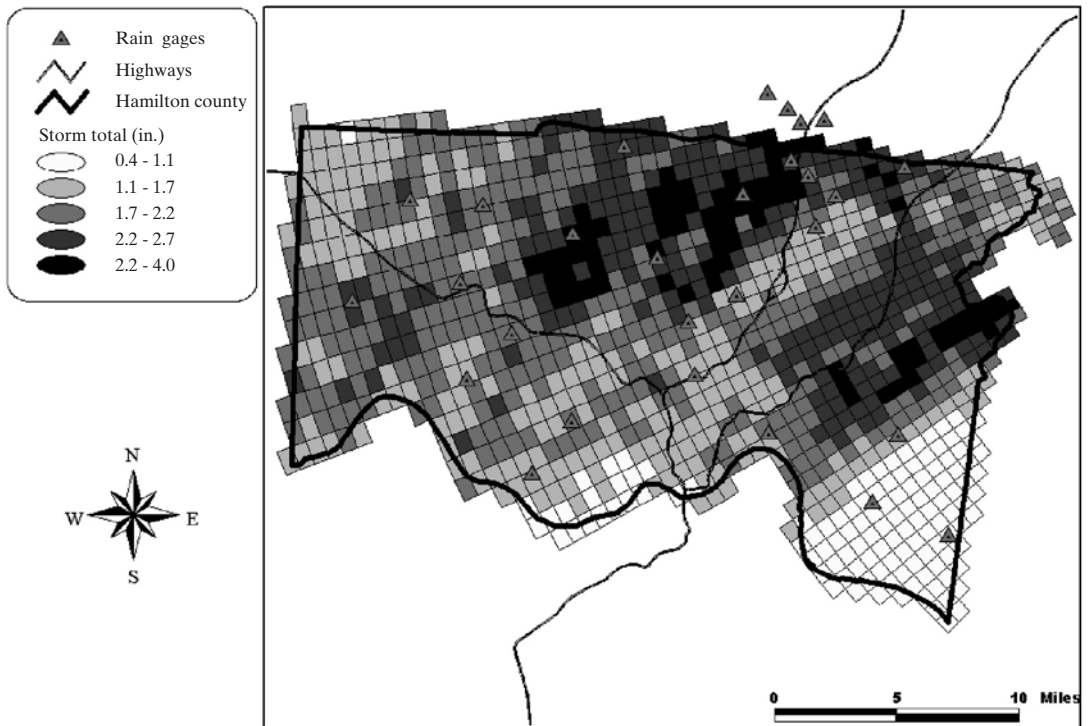
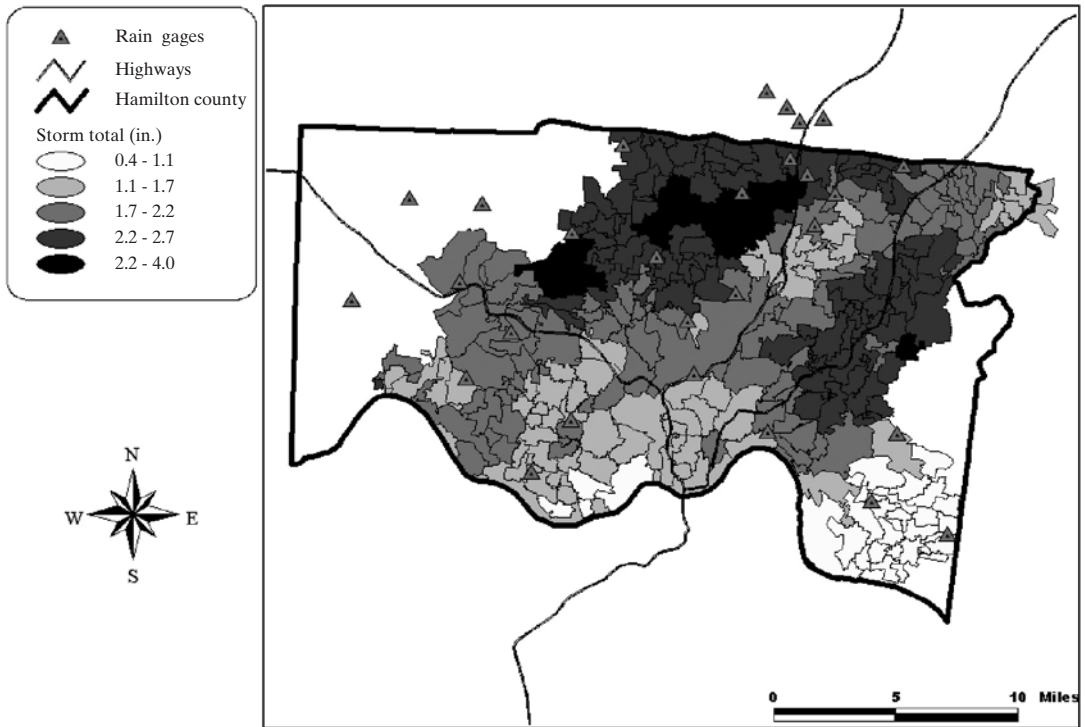


Figure 11–8

Gage-adjusted radar rainfall storm total for August 28, 2006, over Hamilton County and Greater Cincinnati, Ohio. Rainfall distribution is shown in the radar sample volumes in polar coordinates. Rain gage locations are indicated by the triangles.

**Figure 11-9**

Rainfall storm total aggregated by subbasin of the sewer network.

basins are used to correct and adjust the radar rainfall. In some instances, where no rain gages exist directly in the target basin, radar corrected with surrounding gages is the only means of providing accurate and representative rainfall over the target basin.

To produce the rainfall map shown, radar reflectivity is converted to rainfall rate, using a $Z-R$ relationship, and sampled at the same location, as several rain gages, producing radar hyetographs at each gage location, as illustrated by the hyetograph presented in Example 11-3. Adjustment of the radar storm total may be performed using mean field bias adjustment, local bias adjustment, or both. Gages that fall outside established outlier criteria, such as upper and lower confidence intervals of $\pm$ one or two standard deviations, should be excluded in forming the radar product. Techniques for bias adjustment apply a correction factor to enhance the accuracy of the radar rainfall estimates. Here the ratios of gage to radar accumulations (G/R) are spatially distributed over the spatial domain to account for variations in DSD and updrafts/downdrafts during the storm event. The method used here is the local bias correction technique. Figure 11-10 shows the scatter plot of the radar vs. gage accumulations after the radar is corrected using a local bias correction. In

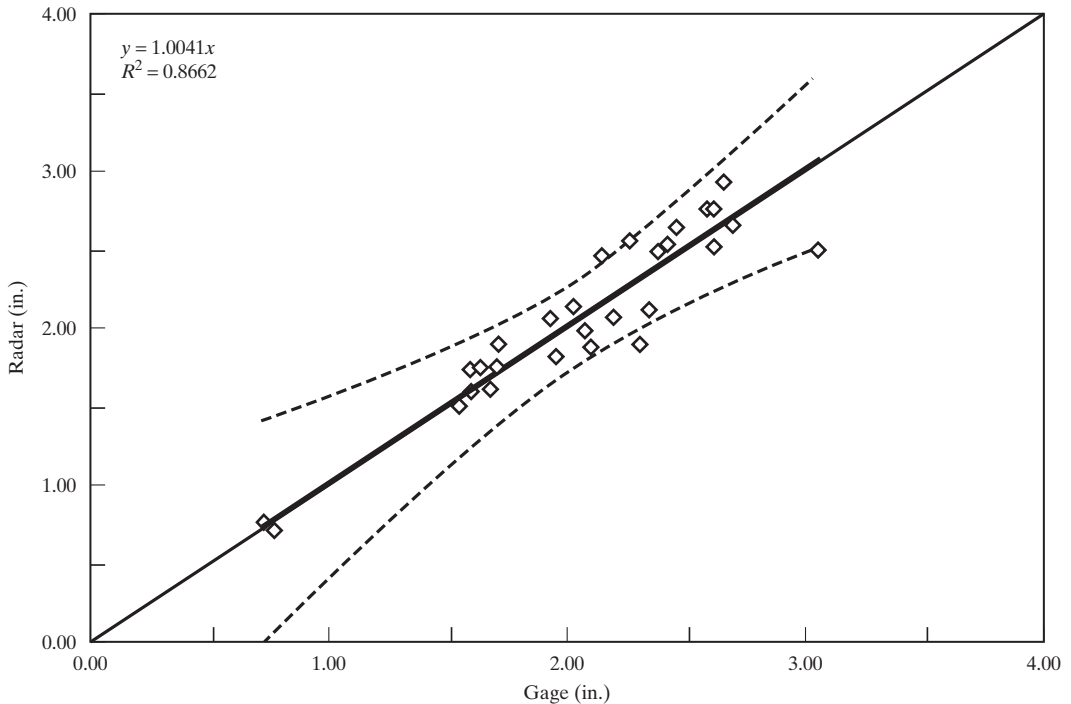


Figure 11-10

Scatter plot of radar and gage rainfall accumulations after local bias correction for the August 28, 2006, event. Confidence intervals are shown here for the 80% level.

this particular event, 28 of 35 gages are used. Of the seven gages rejected, two had missing data and five fell outside the quality control limits established at $\pm$ one standard deviation. The resulting bias-corrected gage–radar rainfall pairs fall within the 80% control limits, as indicated in Figure 11-10.

Data manipulation is usually required before radar input may be utilized with hydrologic models unless specifically designed for in the model. Other than general meteorological applications, unadjusted radar rainfall is usually not sufficiently accurate to be used directly for hydrologic purposes. Processing of the rainfall data produced from radar, besides being necessary for enhancing the accuracy, is needed for transforming the rainfall into a suitable grid or into subbasin aggregations and into file formats that can be read by the model. GIS supports the use and management of radar rainfall data along with the other geospatial data such as digital elevation data, soil maps, and vegetative cover (see Chapter 10). General-purpose GIS software tools, such as ArcView, or special-purpose processing software may be employed for creating the required model inputs. Software systems used operationally automatically transform and “feed” the rainfall data to a hydrologic model or make comparisons to threshold values.

Flash Flood Forecasting

Integration of rainfall monitoring technology produced an efficient and valuable tool for recognizing the threat of flooding with a detailed, site-specific display system. Emergency managers require accurate information on the specific locations and severity of potential flooding. Near-real-time operation of flood warning systems supported by radar rainfall serves emergency managers, helps reduce public exposure to flooding, and increases public safety. Flash flood systems operated in real time help to identify the development of flood hazard conditions that warrant public safety response for roadway closings and building evacuations. Observational sensors using various telemetry methods, e.g., radio or telephone, transmit stream water level and rainfall data to a central site for use in decision-making and for accuracy enhancement of the radar. Operational use of flood warning information systems can improve stormwater management, especially where structural solutions may not be feasible. Implementation and operation of flood warning information systems can complement drainage infrastructure design to provide warnings when runoff exceeds the drainage infrastructure capacity. Such systems rely on radar and rain gage networks for input to hydrologic models to update the evolution of rainfall and runoff and to forecast future rainfall. Distributed models such as *Vflo*TM are useful for operational flood forecasting (Bedient et al., 2003; Vieux et al., 2003), because having accurate rainfall input over the target basins is critical for accurate flood forecasting (Vieux and Bedient, 2004; Vieux and Vieux, 2005; Looper et al., 2009; Looper and Vieux, 2011).

Radar data obtained in real time along with rainfall data measured by rain gage networks provide rainfall input for comparison to rainfall thresholds and for hydrologic modeling (Janek et al., 2006; Looper and Vieux, 2011). The rainfall produced by the remnants of Tropical Storm Hermine September 7–9, 2010, is shown in Figure 11–11 is used to forecast stream locations that are expected to flood, and to predict when and how much flooding will occur for a range of watershed sizes and time of concentration. In Figure 11–12, a zoom-in to Lake and Rattan basin is shown with rainfall averaging 10 inches over Lake Creek and 9 inches over Rattan Creek. The gridded display (see Fig. 11–11) is transformed to subbasin averages (Fig. 11–12) through aggregation. Real-time operation of hydrologic models is supported by the combination of rain gage and radar as input to the distributed model *Vflo*TM. The next section describes distributed models in more detail.

Distributed Hydrologic Modeling

Advances in physics-based distributed hydrologic modeling have made it possible to support real-time decision making for reservoir operations; hydrologic analysis of storm events; long-term water-balance runoff–recharge studies, and prediction of site-specific flooding for stormwater and emergency management. Gridded rainfall and the spatial variation captured by radar can be used in a distributed model directly without lumping to large watershed

Figure 11-11
Storm total from
Tropical Storm
Hermine over
watersheds in and
around Austin,
Texas.

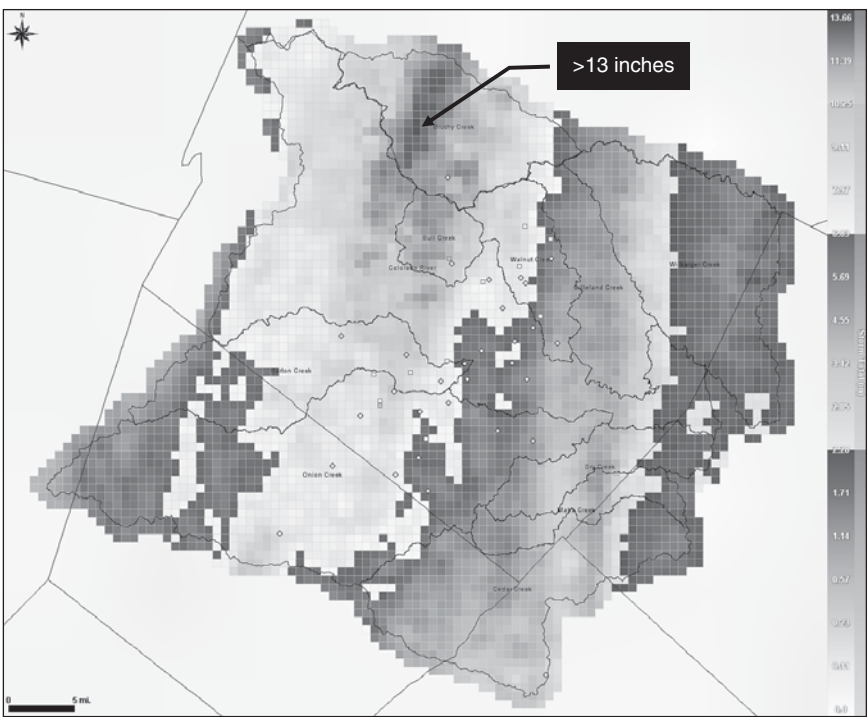
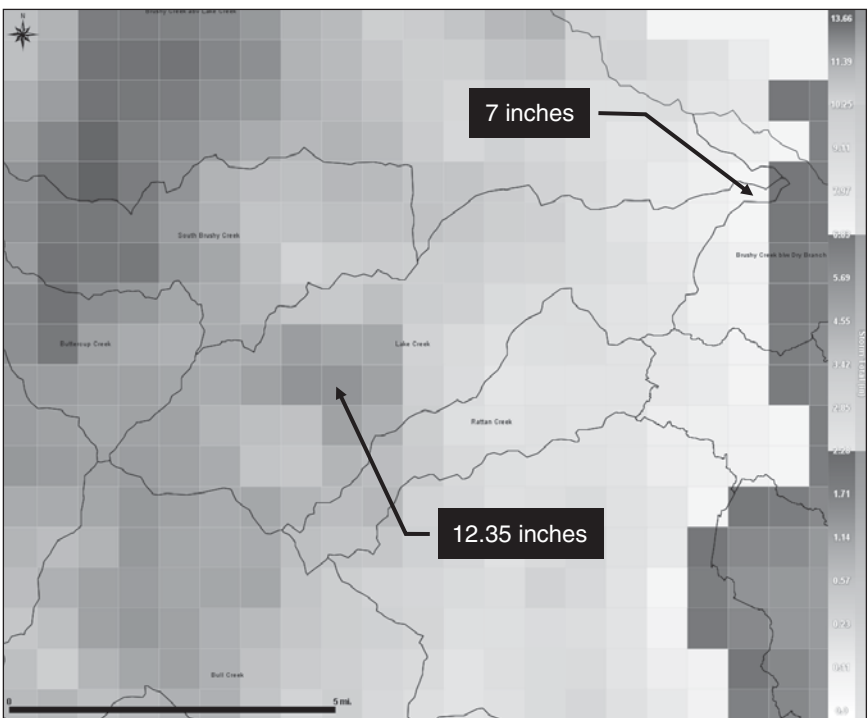


Figure 11-12
Gridded rainfall
over Lake and
Rattan Creek
during Tropical
Storm Hermine.



areas. Evaluation of the hydrologic effects of land use/cover changes can be accomplished with a distributed model. In the application described below, an ungaged watershed is modeled with radar data to identify whether specific land use/cover changes were responsible for flooding, or if it was due to intense rainfall. The linkage between radar and a distributed model occurs where the temporal evolution of the rainfall hyetographs in each model grid cell is supplied by gage-adjusted radar rainfall accumulations for a specified interval, e.g., 5 or 15 minutes. The grid-cell-based distributed hydrologic modeling scheme is shown in Figure 11–13, which shows the grid-cell connectivity based on terrain and the principal direction of slope. Figure 11–14 shows a shaded relief map and hydrograph inset with simulated (solid line) and observed streamflow (dotted line) for Lake and Rattan Creeks in Austin, Texas, during the remnants of Tropical Storm Hermine. The information known for the watershed consisted of LiDAR 1-m terrain, soils from the USDA-NRCS, and land use/cover from NLCD. The radar rainfall used as model input was produced in real time by the radar rainfall system during Tropical Storm Hermine (see Figs. 11–11 and 11–12 above). Remarkably good agreement, without calibration, was obtained with little knowledge about the watershed except parameters derived from available geospatial data and the physics-based equations used to compute runoff. The 2.25 mi² basin has an average hydraulic roughness of $n = 0.029$, average slope of 2.9%, estimated 15% impervious cover, and clay soils with a depth of 28.2 inches, Green and Ampt parameters that are estimated from texture to have a wetting front suction of 28.2 in, and saturated hydraulic conductivity of 0.06 in/hr (Saxton and Rawls, 2006). The channels are represented by cross-sections cut from the LiDAR terrain and used in each 75-m model grid. A feature of physics-based distributed models is that they can be set up using physically realistic model parameters and radar rainfall input, and can produce meaningful results without model calibration as demonstrated here.

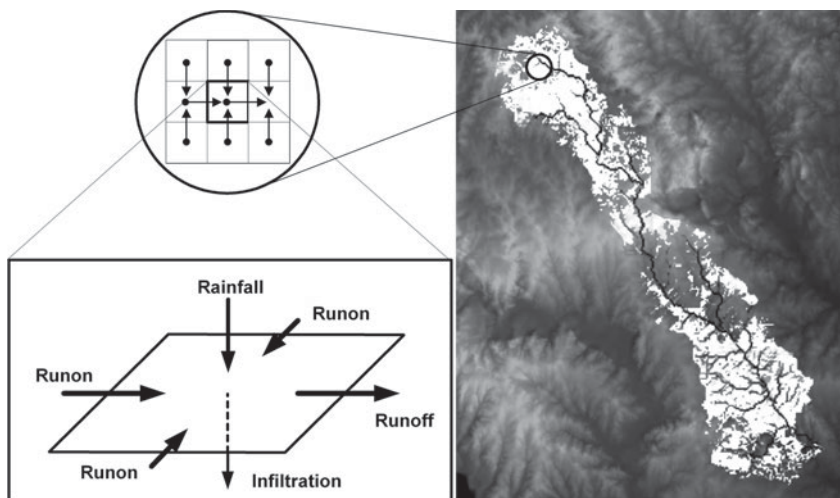


Figure 11–13
Distributed model
runoff scheme and
associated grid
connectivity.

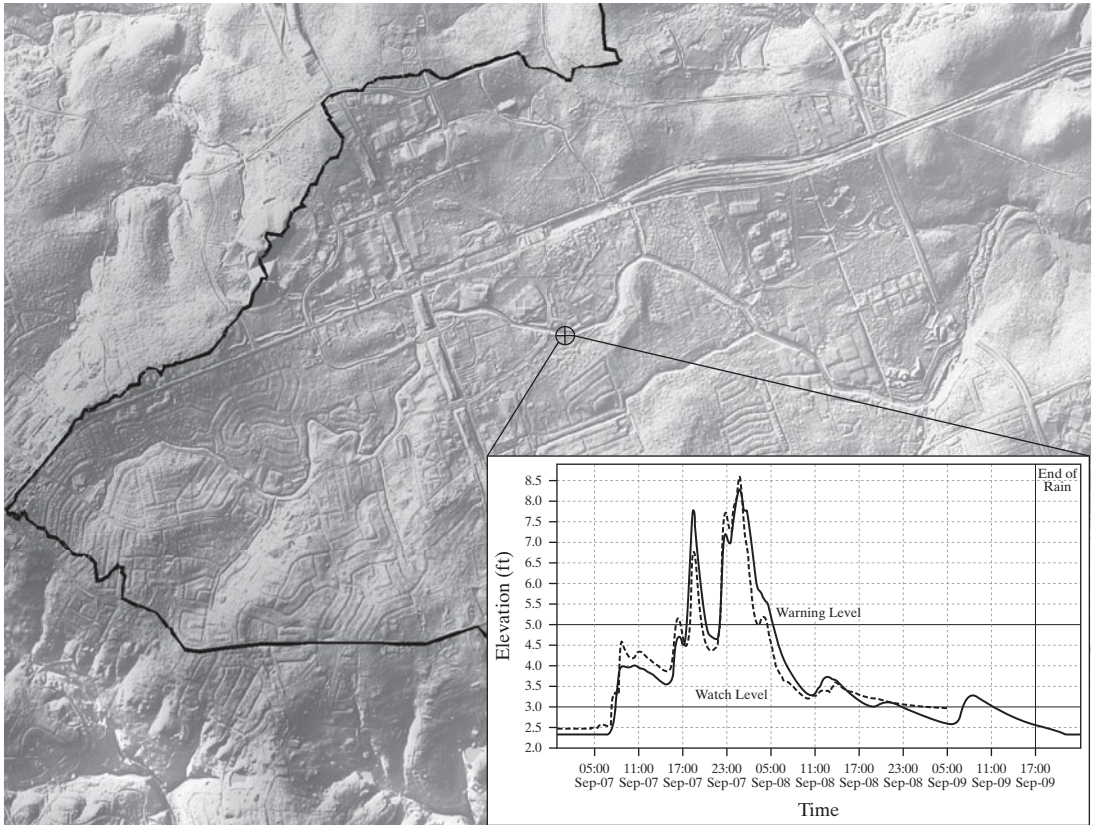


Figure 11-14

Shaded relief map and hydrograph inset (lower right) with simulated (solid line) and observed stage (dotted line) for Lake and Rattan Creeks in Austin, Texas, during the remnants of Tropical Storm Hermine.

Vflo™ Model Background

The physics-based distributed hydrologic model, *Vflo*™, employs a finite-element solution of the kinematic wave equations for runoff routing and Green–Ampt infiltration equations and saturation excess for runoff generation. Physically realistic model parameters are derived from geospatial data representative of the terrain, soils, and land use/cover and input as gridded files from the GIS. Vieux (2004) describes the model background, governing equations, parameterization using geospatial data, distributed model calibration, and sample model applications. Application of the model to predict reservoir inflow and its sensitivity to rainfall input and soil moisture is presented by Vieux et al. (2009). Vieux and Moreda (2003) describe the ordered physics-based parameter adjustment (OPPA) approach used to calibrate a distributed model. The identification of optimal parameters obtained by scaling the magnitude of distributed parameter maps is described in White et al. (2002 and 2003).

The model is designed to use distributed rainfall rates derived from radar and rain gages in real time and in retrospective studies using archival rainfall. Soil moisture is tracked continuously for real-time flood forecasting and streamflow prediction. The model is parameterized with generally available geospatial data that represent the watershed terrain, soils, and land cover/use conditions and impervious areas. The runoff is generated using both saturation and infiltration rate excess. Runoff is routed from overland grids that connect to channels within a drainage network. The kinematic wave equations are solved numerically for the drainage network based on conservation equations. The flow depth, h , depends on runoff from upslope, infiltration, rainfall intensity, slope, and hydraulic roughness. The kinematic wave equation in terms of flow depth is

$$\frac{\partial h}{\partial t} + \frac{S^{0.5}}{n} \frac{h^{5/3}}{x} = i - f, \quad (11-13)$$

where S is slope, n is the Manning roughness coefficient, i is rainfall rate, and f is infiltration rate. The type of runoff represented by this equation is known as Hortonian runoff or **infiltration excess runoff**. Additionally, when the soil profile becomes saturated, rainfall becomes runoff known as **saturation excess runoff**. Both overland and channel flow may be represented using this equation with appropriate modifications. In general, numerical solutions of Equation 11–13 are required since analytic solutions do not exist except with simplifying assumptions. There is numerical dispersion associated with the numerical solution that produces hydrographs that attenuate and approximate diffusive wave solutions. The kinematic wave equations solved in this manner yield useful approximations with diffusive characteristics, but without reverting to the full St. Venant Equations that are often difficult to solve when applied to the wide variety of conditions found in a watershed.

Channel characteristics are represented using a trapezoidal cross section, surveyed cross sections, or stage–discharge rating curves. The model is applicable both in rural watersheds that range in scale from headwaters to rivers and in urban settings by explicit representation of hydraulic characteristics and watershed conditions. The model setup and data required are as follows.

1. **Geospatial data.** Geospatial data are used to set up *Vflo*TM starting with a digital elevation model (DEM) for the derivation of slope and flow direction; remotely sensed land use/cover for overland hydraulic roughness; impervious area and soil characteristics for the derivation of Green and Ampt infiltration parameters and soil depth; channel geometry and hydraulic characteristics.
2. **Precipitation inputs.** Rainfall and snowmelt may be input on a gridded basis as maps, or input as uniformly distributed rainfall rates/accumulations. Distributed precipitation formats supported include radar formats, e.g., NEXRAD Level II reflectivity, and ESRI ASCII grids. Automated filtering of the radar grid into the model grid is

available when geographic projection information is provided. Point hyetographs may be input for subareas such as basins or uniformly over the entire domain with user-defined or SCS-type hyetographs. Snowmelt is modeled on a distributed basis using the temperature index method. Synthetic dynamic (moving) design storms are simulated for defined hyetographs, movement, and direction.

3. **Runoff routing.** The kinematic wave equations are solved on a grid-cell basis within a drainage network composed of finite elements. The formulas are adapted to represent overland and channel hydraulics. Floodplain storage effects can be simulated using the Modified Puls method to account for attenuation due to temporary storage. Complex channel hydraulics can be represented by (1) rating curves for stage area/discharge, (2) surveyed channel cross sections with slope and Manning's roughness, or (3) trapezoidal cross sections with slope and Manning's roughness.
4. **Soil moisture and infiltration.** The infiltration module is the Green and Ampt equation, which relies on the soil properties to model runoff. Infiltration rate excess runoff is computed in each grid cell as a function of soil properties and antecedent conditions. Once the soil storage capacity is filled, saturation excess runoff is then computed. When the soil moisture is modeled over time, the infiltration rate is adjusted to account for a range of soil moisture. The rate of soil moisture depletion is limited by the climatologic evapotranspiration (ET) rate and available soil moisture. Impervious area and initial abstraction may be set to account for urbanization effects or ponding on the land surface.
5. **Distributed output.** A feature unique to distributed models is the capability to output distributed runoff and other hydrologic quantities at any location. This capability supports generation of hydrographs at locations selected in the drainage network regardless of current watershed delineations. Distributed runoff can be used for water quality modeling and transport such as debris flow forecasting where heavy rainfall, soil conditions, and slope play a role in the initiation of landslides and subsequent transport of mud and debris. Because the model outputs stage at distributed locations along the channel, inundation mapping can be accomplished that is more detailed than routing in channel reaches.

The *Vflo*TM model is designed for ingest of radar or gridded precipitation on an event scale, or over a longer-term continuous basis. In the latter case, a series of distributed rainfall grids are provided and used along with potential evapotranspiration to model runoff. Cumulative runoff volume, infiltration volume, and discharge are tabulated during the continuous simulation period. Thus, with continuous simulation, the history of storm event depth, hyetographs, duration, interarrival time, and the effects of evaporation on runoff can be modeled.

Hydrologic Model Summary

HEC-HMS. Two options exist for importing WSR-88D data into HEC-HMS. One method is to take the average rainfall from the radar grid cells within a basin or subbasin and create a pseudo-precipitation gage. The time step can vary from about 5 minutes up to a user-defined increment. HEC-HMS is then run with radar rainfall as input, and output hydrographs are compared to observed stream gage information. The number of subbasins in the HEC-HMS model affects the resulting hydrograph accuracy, because averaging high- and low-intensity rainfall over larger areas can cause discrepancies in the modeled watershed response. Another option in HEC-HMS is to use gridded data in the Modified Clark (or ModClark) unit hydrograph method. In either case, it is important to use the radar data that have been adjusted to gage accumulations, or at least compared for consistency. Import options exist for bringing both of these data types into the HEC-HMS model. Basin hyetographs derived from radar can be input in a manner similar to a rain gage hyetograph.

Flood warning systems. Radar rainfall can provide timely information for estimating river or stream levels related to flooding. The advantage of radar is that rainfall can be extracted directly over basins of interest in near real time. Based on a selected threshold base of rainfall accumulations, a flood threat can be recognized and actions taken to mitigate impacts or reduce public exposure to flooded conditions. Besides computing thresholds, running a hydrologic model in real time using the radar rainfall as input can provide additional flood threat information (Bedient et al., 2003; Vieux and Bedient, 2004; Janek et al., 2006). Lead time and the hydrologic response time of a basin can limit the area that can be feasibly forecast. Soil moisture tracking by the hydrologic model run in real time is an important consideration in most watersheds, such that the model will be properly initialized by antecedent soil moisture conditions. Chapter 12 presents more details on flood alert systems.

Distributed hydrologic models. Distributed models preserve the spatial variation of slope, hydraulic roughness, infiltration, and rainfall input without lumping to the watershed-level of aggregation. Such models use physics-based conservation equations to represent overland and channel flow. Full dynamic, kinematic, and diffusive wave analogies are used to derive systems of equations that are then solved by finite-difference or finite-element methods. *Vflo*TM is a distributed model that runs as a real-time operational application or as a desktop model (Vieux et al., 2003; Vieux and Vieux, 2002; Vieux, 2004). Parameters are derived from geospatial data and published physical characteristics according to the method described in Vieux (2004) and Vieux and Moreda (2003).

Additional Resources

Understanding the usage and variability of precipitation can be investigated using the tutorials provided online for *Vflo*TM. See the website for supplemental resources that include access to model software, tutorials, and required data.

EXAMPLE 11–4

*Vflo*TM MODEL EXAMPLE

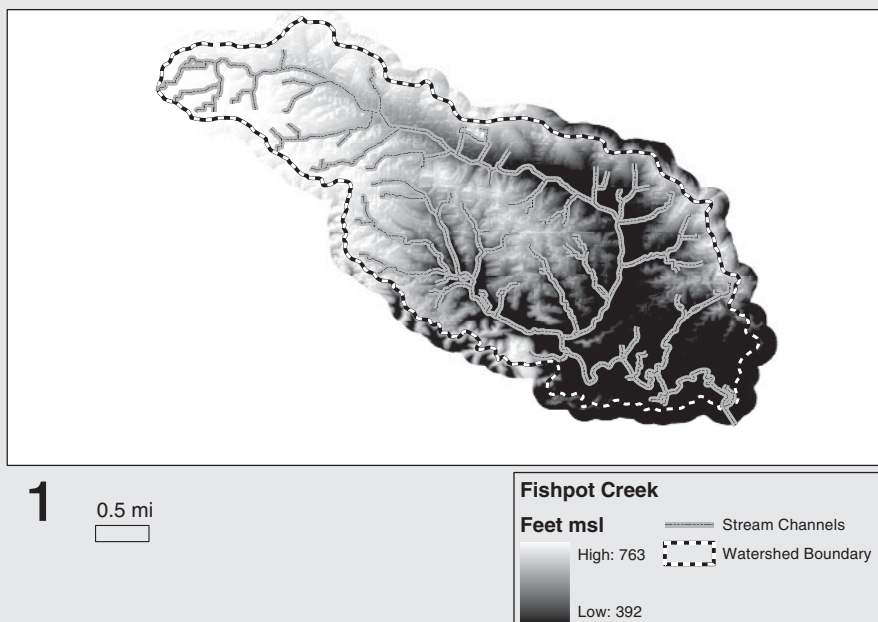
As this example demonstrates, the physics-based distributed approach produces hydrograph response that matches observed flow without model parameter calibration. The *Vflo*TM model uses the conservation of mass and momentum equations to route rainfall excess through a network composed of overland and channel cells. Runoff from both infiltration rate excess (i.e., when rainfall rate exceeds infiltration rate) and saturation excess (when the fillable porosity within the soil profile becomes saturated) is modeled simultaneously in the watershed.

The hydrograph response to rainfall input is determined from the hydraulic characteristics of the drainage network. The rainfall rates in each grid cell determine how much runs off, while the slope and hydraulic roughness determine the flow depth and velocity of flow in overland and channel cells. The configuration of the drainage network that defines the watershed shape and the diversity of hydraulic travel time also determine the hydrograph shape as expected from discussion in previous chapters on watershed response. The distribution in time and space of rainfall intensity coupled with terrain characteristics become an important determinant of the response in terms of runoff volume, peak discharge, timing of the peak, and the entire hydrograph shape.

Accurate portrayal of the distributed drainage network, vegetative cover, and soil characteristics can result in hydrograph predictions that agree well with observed streamflow without model parameter calibration. An example of this agreement is offered for the 9.58 mi² Fishpot Creek located in eastern Missouri for three events, two in May and July of 2008, and one in February of 2009. The model parameters and radar rainfall used in this case study are presented below and are made available via the website that contains detailed datasets for *Vflo*TM and radar rainfall.

Watershed Description

To begin the watershed analysis, digital terrain is obtained from LiDAR presented in Figure E11–4–1, and is used to extract the drainage network and slope. Channels are delineated for 10% of the area. The drainage network extracted by the *Vflo*TM module called AutoBop is seen in Figure E11–4–2. The cells are assigned as channels such that channel cells comprise 10% of the watershed. See Chapter 10 for a description of GIS watershed delineation. The watershed is long and narrow implying that runoff from the upper part of the watershed will take longer to arrive at the outlet than runoff following flow paths nearer to the outlet.

**Figure E11-4-1**

Digital terrain data showing topographic relief and stream channels.

**Figure E11-4-2**

Fishpot Creek outlet and drainage network, overland and channel cells, extracted by Vflo.

Parameters

Once the network has been extracted, parameters are assigned to overland and channels based on land use/cover, or more specific information known for the watershed (see Chapter 10). Maps of hydraulic roughness are derived from standard lookup tables of land use/cover (see for example, Vieux, 2004, p. 124) and used to populate the model grid. Table E11-4-1 presents the range of distributed parameters used to obtain the three event hydrographs.

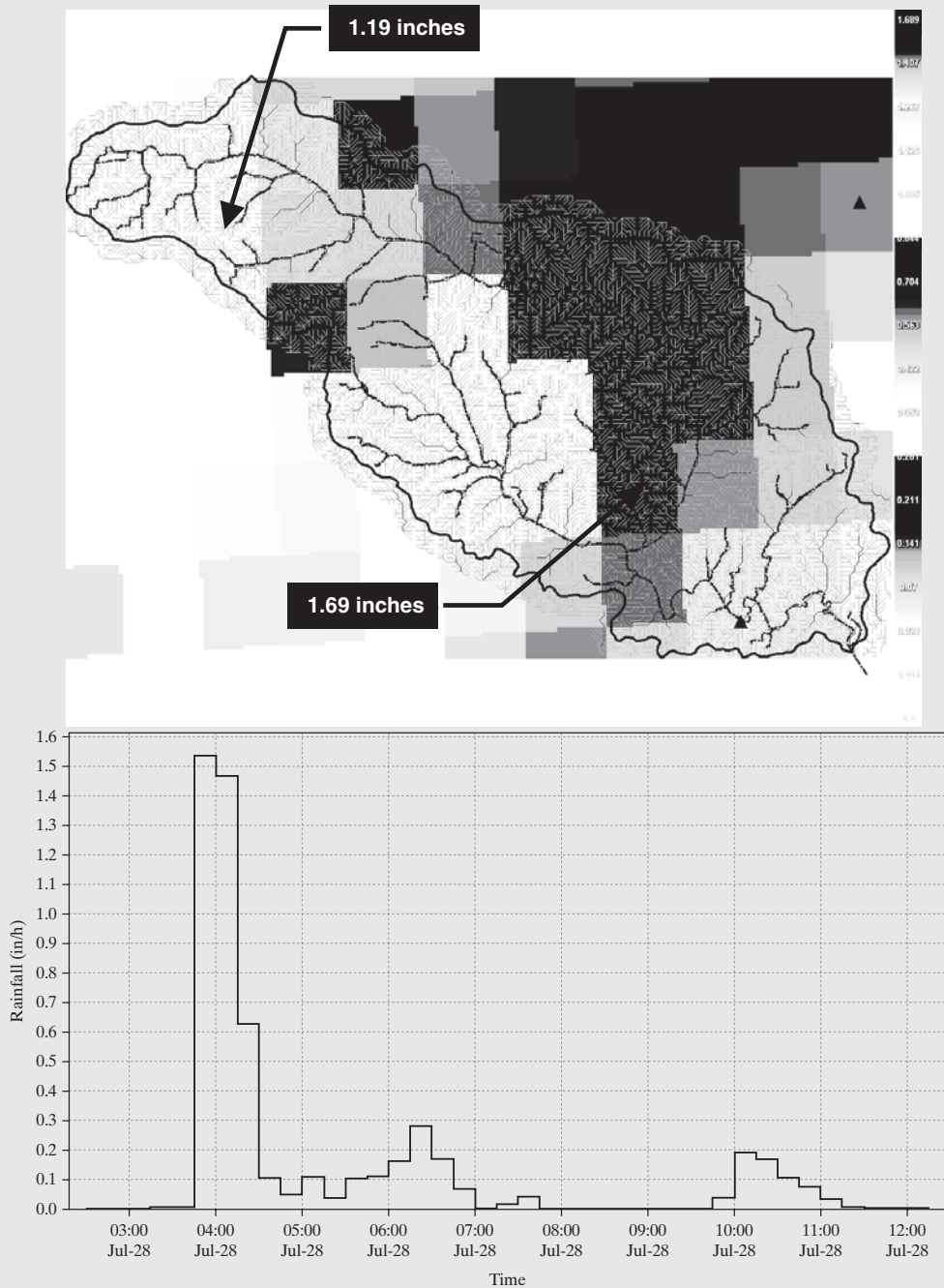
The soils in the Fishpot Creek are classified as a silt loam in the top layer, 0–7 inches, (A-horizon) with urban built-up areas for over 60% of the soils in the watershed. The clay accumulation in the B-horizon of the soil transitions to silty clay loam at deeper depths. From sand, silt, clay, and gravel fractions, the Green and Ampt parameters of hydraulic conductivity, wetting front suction, and porosity are estimated for the A-horizon of the soil profile. Urban development is evident from the patterns observed in the impervious map seen in Chapter 10, Figure 10-5(a). For this watershed, imperviousness is estimated from the NLCD land use/cover and averages 0.31 (31%). Converted the NLCD to roughness, the watershed ranges from 0.024 to 0.2, with an average of $n=0.040$ in the watershed. Both the Green–Ampt and hydraulic roughness parameters in Table E11-4-1 are sampled from the original maps into the 75-m model grid for the basin.

Radar Rainfall Input

Inputs to the model consist of gauge-adjusted radar rainfall (GARR); that is, rain rates have been adjusted by the local bias correction described herein Chapter 11. Three events are tested using GARR and the output simulated at the USGS 07019120 stream gauge and compared to observed flow during this events. In Figure E11-4-3, the rainfall total (upper image) and the basin-averaged hyetograph (lower) are shown for the July 28, 2008, event. The total rainfall for this event averages 1.37 in. with the main runoff producing rain rate over 1.5 in./hr falling between 3:30 and 4:30 across the basin.

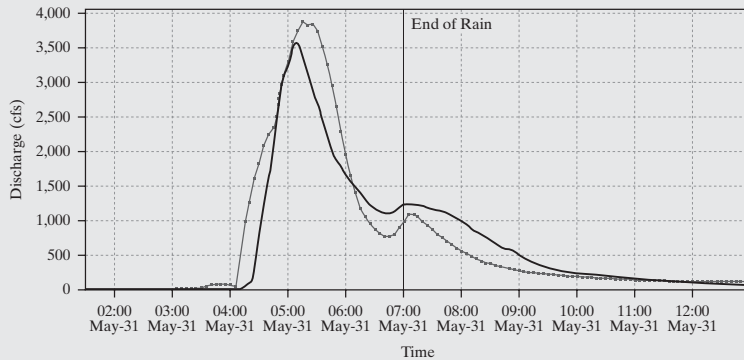
Table E11-4-1 Range and Mean value of Distributed Parameters

Parameter	Min.	Mean	Max.
Roughness	0.024	0.040	0.200
Slope	0.07%	6.626%	37.46%
Hyd. conductivity	0.05 in/h	0.09 in/h	0.13 in/h
Wetting front	12.36 in	15.7 in	19.94 in
Effective porosity	0.423	0.447	0.455
Soil depth	2.0 in	4.0 in	5.0 in
Initial saturation	0.95	0.95	0.95
Impervious	0.0	0.316	1.0
Abstraction	0.0 in	0.0 in	0.0 in
Channel width	50.0 ft	50.0 ft	50.0 ft
Side slope	1.0	1.0	1.0

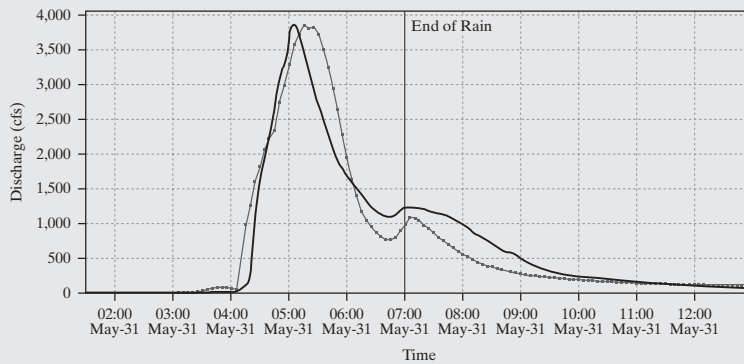
**Figure E11-4-3**

Rainfall storm total (upper) and basin hyetograph (lower) for the event on July 28, 2008.

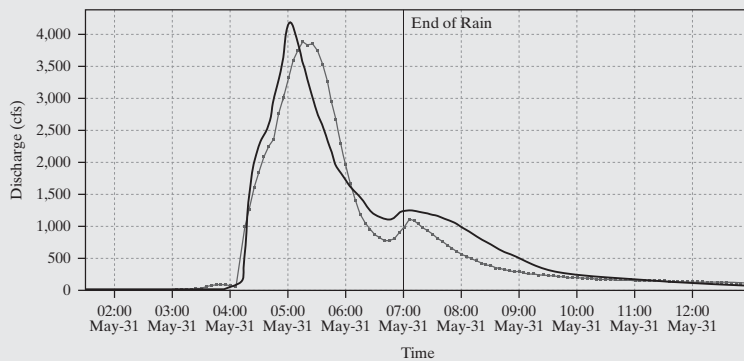
Initial degree of saturation is taken from NOAA soil moisture. The initial degree of saturation (IS) is varied $\pm 5\%$ and then simulated and compared to observed discharge. No adjustment is made to the *a priori* model parameters for the three events shown. The model parameters are assigned based on standard lookup values for hydraulic roughness from land use/cover and soils geospatial data (See Chapter 10). Figures E11-4-4, 4-5,



IS = 80%



IS = 85%



IS = 90%

Figure E11-4-4

Hydrograph response for event on May 31, 2008, showing sensitivity to initial degree of saturation.

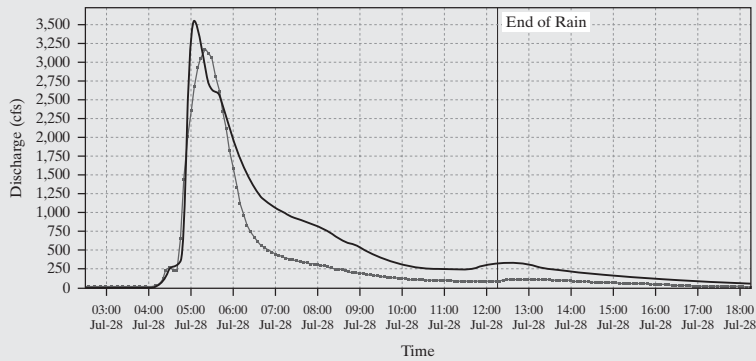
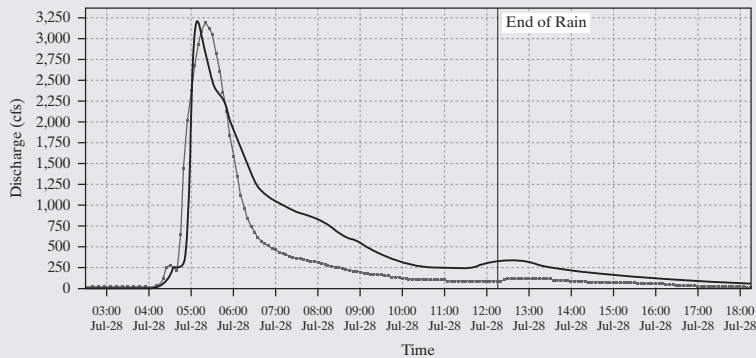
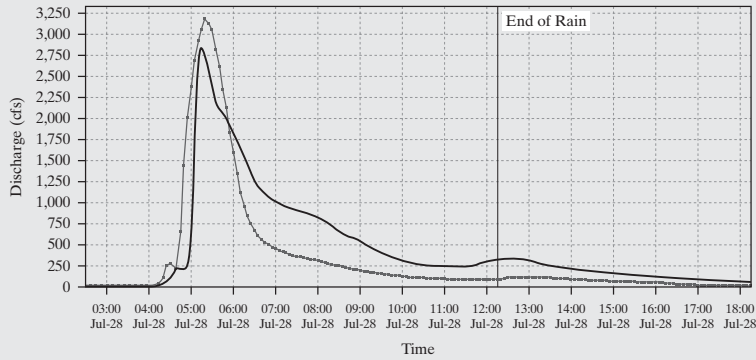
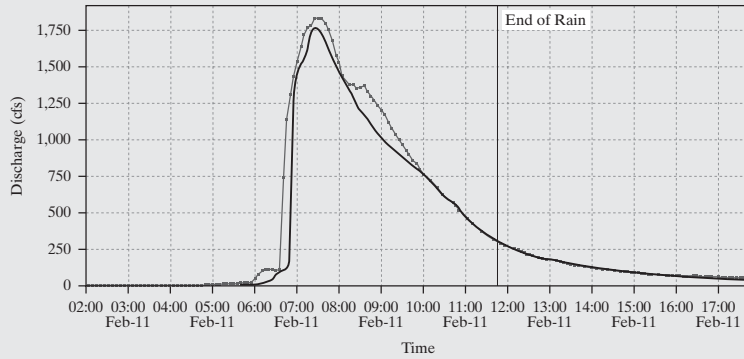


Figure E11-4-5

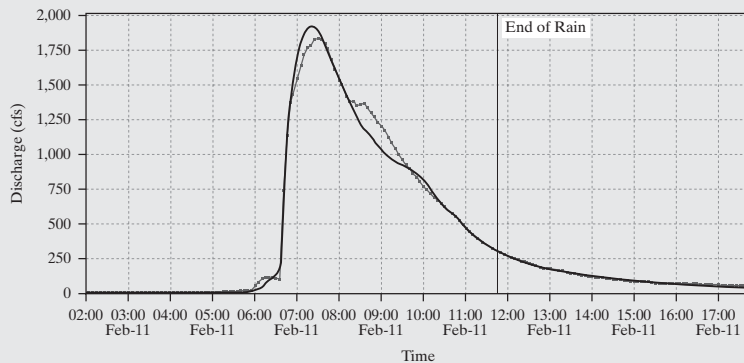
As in Figure E11-4-4, but for event on July 28, 2008.

and 4-6 show the event hydrographs for each initial saturation on May 31, 2008, July 28, 2008, and February 11, 2009, respectively.

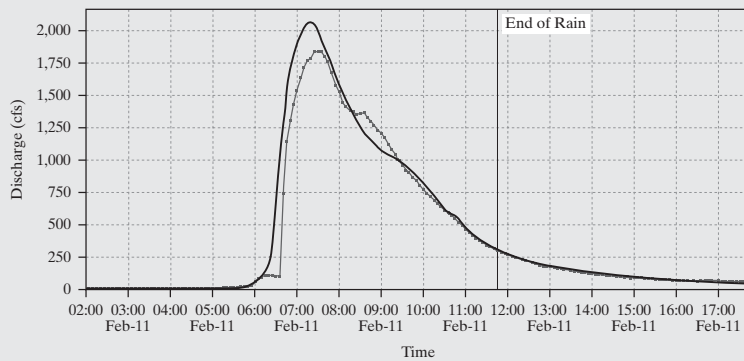
In summary, this example demonstrates that good agreement can be obtained for a set of storm events using a single model parameter set, and without calibration. In the model *Vflo*TM, parameters are derived from geospatial data with typical values of roughness and infiltration assigned. The GARR input plays a role in these results, because accurate portrayal



IS = 80%



IS = 85%



IS = 90%

Figure E11-4-6

As in Figure E11-4-5, but for Event on February 11, 2009.

of rainfall variability in space and time is critical to obtaining an accurate hydrograph response for actual storms. Varying the initial degree of saturation affects the infiltration rate defined by the Green and Ampt soil parameters, and results in increased/decreased change in peak discharge and runoff volume. The hydraulics of the channel and overland elements define the hydrograph response that is very close to observed flow.

SUMMARY

A primary advantage of weather radar is the density of measurement that is not obtainable by rain gage alone. Combining these two sensor systems produces better rainfall estimates that are more accurate and characteristic of rainfall over a watershed. Radar adapted from historical military uses has provided a valuable tool of detecting severe weather and can be adapted for hydrologic purposes. Using radar, the hydrologist benefits from having more information about rainfall rates at high resolution in space and time over large areas. The way in which radar measures rainfall rates depends on assumptions about the number and sizes of raindrops in a representative volume of the atmosphere. The number of drops and distribution of sizes are described by the probability distribution of drop sizes. It was shown that both radar reflectivity and rainfall rate depend on the parameters of this distribution. The relationship that embodies this dependence is known as the *Z-R* relationship. Various *Z-R* relationships have been derived from a theoretical or empirical basis. Depending on storm type and the power of the radar, a range of *Z-R* relationships is possible. Once an appropriate *Z-R* relationship is selected, comparison with rain gage accumulations is used to remove any systematic error, known as bias. After bias removal, differences between radar and gage estimates remain as random error.

Dramatic progress has been made in the recent decade toward a hydrodynamic approach to hydrologic prediction and the necessary precipitation inputs. Hydrologic models, both distributed and lumped, benefit from having accurate and representative rainfall inputs. Distributed models are particularly well suited to utilizing radar rainfall such as *Vflo*TM that is demonstrated to produce stage and discharge hydrographs with no model calibration. Distributed models designed from the outset to utilize high-resolution rainfall rates from multiple sensors (radar, satellite, and rain gage) allow detailed predictions at almost any location in a watershed characterized with geospatial data relating to topography, soils, and land cover. Besides hydrologic design, radar rainfall can also provide timely inputs to flood warning systems, so that individuals, hospitals, businesses, and emergency management personnel can take proactive steps to protect property from flooding. Considering the need for precipitation measurements for hydrologic analysis and prediction, radar rainfall is clearly one of the most significant advances in hydrology. With the advent of distributed precipitation measurements, distributed hydrologic modeling can be exploited to its full potential in analysis and prediction.

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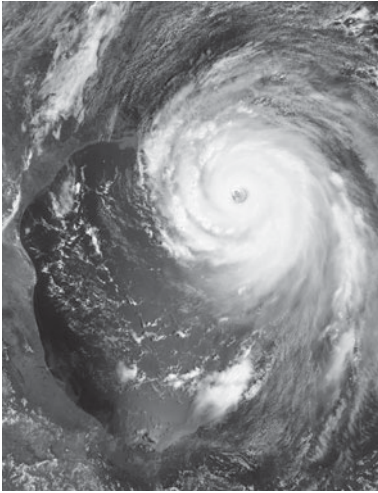
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Chapter 12

Severe Storm Impacts and Flood Management

—By Jude A. Benavides, Ph.D., and Heather M. Winter



Hurricane Katrina on its devastating approach towards New Orleans and the Mississippi Gulf Coast. (Photo courtesy NASA.)



Drainage outlet with gage that monitors water flow.

The overarching theme of this chapter is the importance of a sound floodplain management strategy in mitigating severe storm impacts. In particular, the chapter focuses on the increasing impact of severe storms on major coastal urban centers in the United States. The basic fundamentals and terminology associated with floodplain management and flood damage reduction strategies are introduced and illustrated through a variety of case studies and historical storm events.

Sections 12.2 and 12.3 provide essential information about floodplain management issues and flood damage reduction alternatives, including both structural and nonstructural options. Sections 12.4 and 12.5 discuss the flood control paradox and the growing susceptibility to flood damage caused by severe storms, particularly in areas with rapid, unchecked growth and development. These sections also discuss recent severe storms (Hurricanes Katrina and Ike) and their impact on coastal communities. Section 12.6 discusses the continual need for revamping outdated flood management strategies,

12.1 INTRODUCTION

as well as the need for better planning and enhanced emergency response to severe storm events in both the short and long term. The chapter concludes with a brief case study highlighting the use of flood warning systems to reduce flood-related damages in Houston, Texas.

Severe storms and the resulting major floods have taken their toll in the United States, especially over the last 25 years. Figure 12–1, created by the National Climatic Data Center, shows the distribution of 67 various types of natural disasters with damages in excess of 1 billion dollars that occurred between 1980 and 2005. The image is dominated by hurricane- and flood-related occurrences, particularly in the southeastern and south central regions of the country. The vast majority of these disasters occur in or near highly urbanized cities and coastal areas. Notable among them are Hurricanes Katrina (2005), Ike (2008), Andrew (1992), the midwest floods along the Mississippi (1993), Hurricanes Rita (2005), Charley (2004), and Ivan (2004). The damages for these events alone totaled over \$350 billion. Other, less costly, but still significant flood-related damages can be seen in other portions of the United States. The figure clearly demonstrates the severity of the flood problem at the national level, even after years of investment in “flood control” and the improvement of severe weather forecasts.

12.2
FLOOD
MANAGEMENT
ISSUES
AND BASIC
TERMINOLOGY

Flooding and its associated damages have been with us throughout history. While there are innumerable definitions for the words *flood* or *flooding*, after the major midwest flood of 1993 a fairly complete definition was provided by the U.S. Executive Office of the President (USEOP). The USEOP defined a flood as “a general and temporary condition of partial or complete inundation of normally dry land areas from the overflow of river and/or tidal waters and/or the unusual accumulation of waters from any source that may be associated with undesirable effects to life and property” (USEOP, 1994). The definition emphasizes certain critical aspects of flooding such as inundation and damage, aspects which will be defined and highlighted throughout this chapter.

A **floodplain** can be defined as low lands adjoining a channel or a river, stream, watercourse, or lake that have been or may be inundated by flood-water. Figure 12–2 shows a typical floodplain as the area of inundation to be expected, on average, once every 100 years. Most major channel and river systems have floodplains defined by the Federal Emergency Management Agency (FEMA) through the National Flood Insurance Program (NFIP), as well as by the U.S. Army Corps of Engineers (COE). The **floodway** is an area within the floodplain that contains the fastest-moving water. It is computed by modeling encroachment on either side of the channel such that the floodplain elevation increases by one foot. This is normally computed with a model such as HEC-2 or HEC-RAS (see Chapter 7). The area between the floodplain and the floodway is called the flood fringe.

Billion Dollar Weather Disasters 1980 - 2006

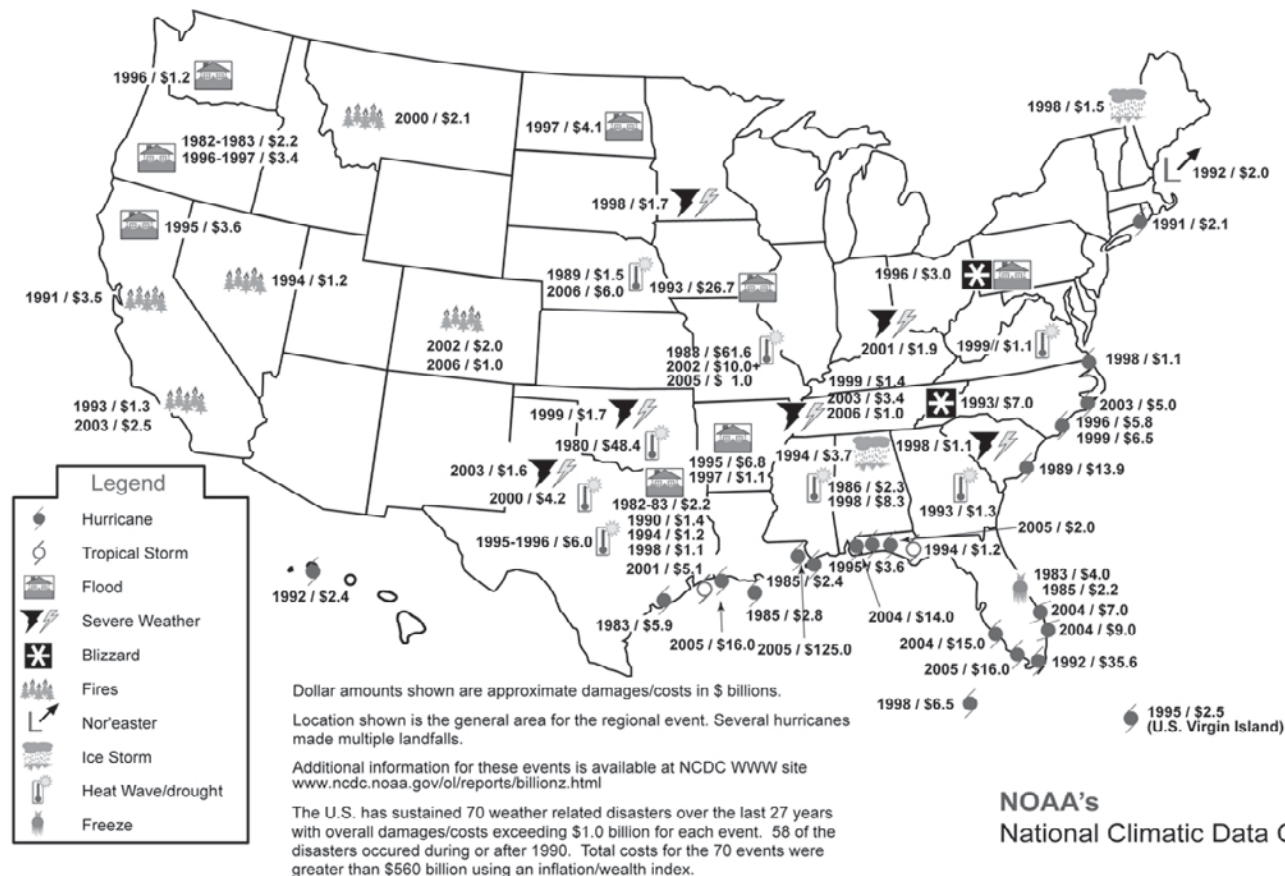


Figure 12-1

Billion-dollar weather disasters in the United States from 1980 to 2010. (Source: National Climatic Data Center, 2005, <http://www.ncdc.noaa.gov>.)

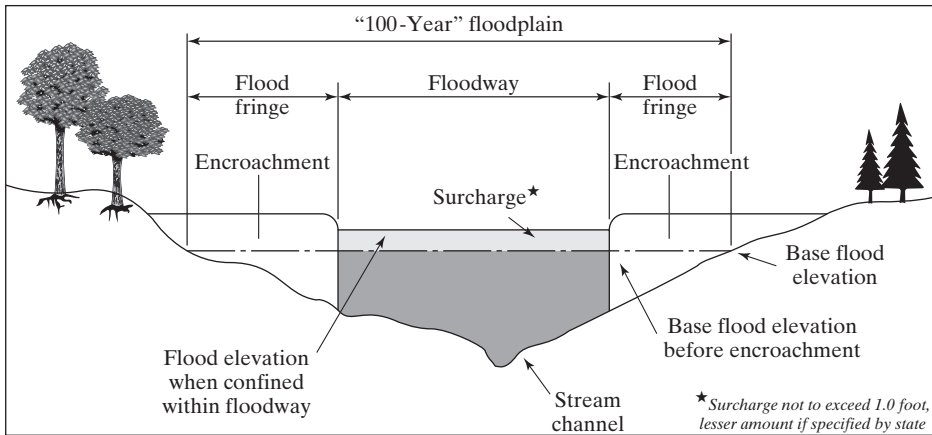


Figure 12-2

A 100-year floodplain and floodway schematic. (Source: National Wildlife Federation, 1998.)

A schematic of a 100-year floodplain and its associated floodway is presented in Figure 12-2. Significant development within a floodplain results in significant losses in water resources, ecological resources, and human resources that may easily outweigh the property value of the floodplain. The floodplain includes a natural means of flood and erosion control that reduces sedimentation, provides large amounts of storage, and naturally reduces flood peaks and valleys. With respect to water quality, the floodplain can filter nutrients and impurities from runoff prior to deposition in the river and serves as a natural temperature and organic waste moderator. From the ecological perspective, floodplains increase biological productivity and provide important fish and wildlife habitats. The integrity of the existing ecosystem is maintained by a sustained biodiversity and an atmosphere conducive to plant and animal growth. Often, these areas serve as natural habitats or wildlife corridors for endangered species. Floodplains represent a wealth of human resources by providing prime locations for aquaculture, enhanced agricultural lands, and open space that can be used for public recreation or nature appreciation.

Floodplain management, a term often incorrectly used interchangeably with *flood control*, is an overall decision-making process whose goal is to achieve the appropriate use of the floodplain. *Appropriate use* is any activity compatible with the risk to natural and human resources located in the floodplain. It is the operation of an overall program of corrective and preventive measures for reducing flood damage, including watershed management practices, emergency preparedness plans, floodplain management regulations, and flood control works. Floodplain management incorporates many of the political, social, and economic factors that directly affect the severity of floods with respect to the damage and choices involving what methods to employ in limiting this damage (USEOP, 1994).

Deciding which flood control alternative to use in any specific watershed is a very important decision within the larger framework of floodplain management. Any general flood control approach might include structural methods, nonstructural methods, or a combination of both. Examples of structural methods include channel modifications (channelization), reservoirs, diversions, and levees or dikes. Those methods that seek to modify the damage susceptibility of developed regions within a floodplain are classified as nonstructural. Nonstructural measures include floodplain land acquisition (also commonly referred to as voluntary relocation or property buyouts), land use restrictions (zoning), flood proofing, flood warning systems, drainage maintenance programs, and public awareness or information programs.

One of the most beneficial aspects of a nonstructural approach to flood control is its greater utilization of the benefits provided by the natural floodplain discussed earlier in this section. However, economically quantifying the total value of these benefits has proven to be a very difficult task. Accurately valuing environmental amenities continues to be the subject of much debate (NRC, 1999a). It is generally agreed that an established standard for assigning a dollar cost to specific amenities, such as wetlands, bottomland hardwoods, riparian forest, and green space, would provide a much-needed quantification of the indirect benefits provided by nonstructural measures. However, there are often too many widely differing variables from location to location to make the establishment of a general, and thus easily implemented, standard (or metric) very likely. Nevertheless, a variety of methods to measure environmental costs and benefits have been established, such as contingent valuation, the benefit-transfer method, and hedonic pricing (NRC, 1999b).

Nonstructural approaches are not new, and there are several outstanding success stories where flooding has been controlled without major channelization or other major structural controls. The Woodlands near Houston (Bedient et al., 1985) and Indian Bend Wash in Scottsdale (City of Scottsdale, 1985) are two excellent examples where recreational areas and detention ponds were used to better manage the floodplain, as urban development continued. However, some of the difficulties previously mentioned have limited the widespread implementation of these methods.

Structural Methods for Flood Damage Reduction

The United States has pursued, throughout much of its history, the mostly singular course of attempting to control floods through the use of structures (Reisner, 1993; Smith and Ward, 1998). The structural approach's rise to dominance began in the 1930s with the passage of the 1936 Flood Control Act and the creation of the Federal Crop Insurance Corporation. This action placed the federal government in the lead with respect to the nation's efforts to control flooding as well as the compensation of its victims. By 1940, the

12.3 STRUCTURAL AND NONSTRUCTURAL METHODS OF FLOOD CONTROL

government had assumed the full cost of building and maintaining dams, channel modifications, and rectification projects for all navigable waters of the United States (USEOP, 1994). Advances in the fields of fluid mechanics, hydrologic systems, statistical hydrology, infiltration, evaporation analysis, and flood routing throughout the 1940s made the goal of designing and constructing large-scale control structures well within the reach and extremely effective (Chapter 1).

Structural alternatives include channel modifications in a variety of forms: the use of reservoirs or detention/retention ponds to retard or contain flood flows; the construction of levees or floodwalls to contain flows within a designated floodway; and diversions to redirect flood flows to an alternate, off-channel floodway or detention pond. Each of these methods is viable under certain conditions and has associated advantages and disadvantages (Mays, 2001). The failure of levees in New Orleans during Hurricane Katrina exemplifies the importance of choosing appropriate structural methods to protect from all reasonable conditions. A brief description of these types of structural alternatives follows.

Flood Control Reservoirs

Flood control reservoirs provide an effective means of managing and controlling flood flows. These basins can range from as simple a structure as a small pond planned alongside a highway or road culvert to a large reservoir with control structures designed to hold extreme volumes of water. Regardless of their size, all reservoirs serve the same function of providing additional storage to attenuate the peak flow experienced during a storm event (Chapter 9).

Flood control reservoirs can be divided into two basic types, detention and retention. Detention reservoirs or ponds are designed to hold runoff for a short period before releasing it to the natural watercourse. Retention ponds are designed to hold runoff for extended periods, usually for aesthetic, agricultural, consumptive, or other uses (Chow et al., 1988). Chapter 9 includes an example of a standard detention pond design used in small urban watersheds. Extremely large retention structures are also referred to as storage reservoirs. These storage facilities are associated with large dams and are often not created solely for flood protection but also for water supply.

The beneficial reduction of peak runoffs and resulting flood protection afforded at a particular point on a stream by a reservoir depends primarily on the fraction of the watershed area above the protected point, which is governed by the location of the reservoir structure. This reduction is progressively reduced as runoff from the watershed area below the reservoir becomes appreciable and the distance downstream of the reservoir increases. This distance varies and is dependent upon watershed characteristics such as the number and size of tributaries and the shape of the watershed. To be effective, a reservoir site must be available not too far upstream. This may

serve as a significant advantage over other types of flood control, since a smaller reservoir may be constructed just upstream of a flood-prone location, providing adequate flood protection while limiting excavation costs and environmental impact.

However, reservoir location may serve as a disadvantage when large-scale protection is desired and location sites are limited, since extremely large reservoirs are required to provide significant downstream protection. The reservoir option provides particularly limited protection against large storm events in areas where both the overland slope and natural channel are small (USACE, 1996). Other significant drawbacks include finding locations to dispose of excavated material and aesthetic concerns when the constructed pond is not being used for flood control (see also Chapter 9).

Channel Modifications and Environmental Impacts

Channel modifications are performed to improve the conveyance characteristics and carrying capacity of a natural stream. A natural channel's conveyance and capacity can be improved by altering one or more of the dependent hydraulic variables of channel slope, depth, width, and roughness (Brookes, 1989). The resulting increased hydraulic efficiency of the "improved" channel results in increased flow velocities, which in turn result in reduced flood stages or water surface elevations for a given storm event.

These modifications, often referred to as "channelization," have been extensively used throughout the United States for decades with effective results (Smith and Ward, 1998). This technique is especially common as a flood control measure for meandering streams of the south and southeastern coastal plains (Brookes, 1988). However, the channelization approach has always been clouded in controversy, since in certain cases the associated harmful effects on the environment may exceed benefits. Channelization permits the control of flooding in natural, existing waterways to protect already established urban, agricultural, and industrial developments. In addition, channel projects allow and often promote economic development within the floodplain. Specific channelization projects may be undertaken for one or more reasons: for flood control, to drain wetlands, to improve navigation, and to prevent bank erosion and channel migration, thereby protecting neighboring property (USACE, 1996).

Channelization has brought major benefits, both direct and indirect, to urban, agriculture, transportation, and other sectors of the economy by providing effective flood control throughout the stretches of river reach where it has been applied. Its effectiveness is highlighted by the fact that from 1820 to 1970, more than 200,000 miles of the nation's waterways was modified (Schoof, 1980).

A history of proven results seemed to make channelization the alternative of choice for many flood control agencies until the mid-1970s. However, growing concerns over the long-term effectiveness of channelization and the

environmental impacts generated by this approach brought the widespread construction of these types of projects to a near standstill. The channelization work of the Soil Conservation Service (SCS) and the COE became so controversial during the late 1960s that the U.S. Congress commissioned Arthur D. Little, Inc., to make a comprehensive nationwide survey of the environmental effects of these projects and report their findings to the Council on Environmental Quality. This landmark report would serve as the basis for extensive debate over the best course of action for flood control throughout the next few decades; however, there can be no denying the near-immediate and broad-reaching impact this report would have on channelization projects (A. D. Little, 1973).

The Little report in 1973 stated that the direct benefits of the channelization projects it studied were, in fact, conservatively estimated. It concluded that the vast majority of flood channel modifications were performing as designed and that the approximate \$15 billion invested, to date, nationwide in channel modifications for flood protection was reducing annual damages by \$1 billion. Some projects, however, were underdesigned and failed to provide adequate flood control effectiveness for their original design storm. Channel projects near Houston, such as the Brays and White Oak bayous, were originally designed to handle large storm events, but capacities were greatly reduced as the development of the floodplain was encouraged (Hoblit et al., 1999). Another serious issue, seen recently in Houston but also in other large cities, is the net effect of channelization on downstream communities. As flows are channeled more quickly with development, they often increase the peak flows from upstream tributaries and exacerbate the conditions downstream. Other channel modifications were severely overdesigned, ill conceived, or otherwise not properly planned and engineered so that the significant environmental impacts far outweighed the flood control benefits they provided.

The Little report indicated that many environmental effects of channel work must be carefully considered before implementing any channelization scheme. These include:

- Drainage of neighboring wetlands;
- Cutting off of oxbows and stream meanders;
- Lowering of ground water levels, resulting in reduced stream recharge;
- Clearing of floodplain bottom-land hardwoods and destruction of wildlife habitat;
- Increased erosion of stream banks, leading to stability problems;
- Increased downstream deposition of silt and sedimentation;
- Increased downstream flooding as a result of increased upstream velocities;
- Impacts on the stream's and receiving body's aquatic life;
- Reduced aesthetics and loss of visual amenity.

The Little report's findings shifted the focus of future research in the 1970s toward the development of improved science and technology to quantify the extent of the environmental impacts of channelization. Through the next two decades, scientific research concerning the impacts of channelization ranging from hydraulic to environmental appeared in a wide variety of publications.

Diversion Channels, Levees, and By-Pass Channels

Diversions are used to reroute or bypass a portion of the flood flow away from a flood-prone area. This results in reduced peak flows at the area of interest. The method only provides downstream protection, and the amount of protection diminishes downstream in a similar fashion to reservoirs. Diversions are often used in conjunction with off-line (away from the main channel) reservoirs, as the diverted flows must still be contained and/or passed around the area of concern. Their significant drawback is that sufficient right-of-way must be available to contain or direct the diverted flows into neighboring basins.

Levees are linear structures, often consisting of earthen embankments or reinforced concrete walls, built parallel to the main stream with the purpose of containing a specified design overbank flow. These structures are often located very near the natural floodway and on both sides of the river, providing protection to the entire natural floodplain. This alternative is considered the oldest and most widespread type of localized flood defense worldwide, including the United States, and plays a fundamental role in flood control. Levees increase the local carrying capacity of the channel and are intended to prevent all flood damage to the adjacent river corridor until the water level exceeds the top of the structure. Their most significant drawback is that if overtopped, flood damage is incurred as if the structure did not exist. Given the historical trend of development within the floodplain (especially with the additional sense of security provided by the floodwalls), damages often exceed what would have occurred without the levee. Additional drawbacks include increased downstream flow rates and resulting increased downstream flood damages, significant right-of-way requirements, notable environmental impacts similar to channelization, and others (NWF, 1998). A rare combination of weather events led to the midwestern flood of 1993 that breached many levees along the Mississippi River.

The floods of 2011 were the newest in the series of floods that has caused to the Mississippi basins. Early estimates were that damage costs could reach around \$4 billion dollars. Much of the damages were caused to crop lands that were inundated from the opening of spillways in order to save cities such as Memphis, Tennessee, and New Orleans, Louisiana. In contrast to the 1993 floods, the 2011 event had the added component of the melting of heavy winter snows in addition to the excessive rainfall (Drye, 2011). The U.S. Army Corps of Engineers decided to open the three post

1927 flood spillways: the Morganza Spillway located 186 miles upriver from New Orleans, the Bonnet Carre Spillway located about 33 miles above New Orleans, and the Bird Point Spillway located in Missouri. This is the first time in history that all three of these spillways have been open at the same time and was done in order to save New Orleans from possible floods that could have been worse than those caused by Hurricane Katrina in 2005 (Rioux, 2011).

More recently, the discussion of levees returned to the forefront of public discussion after Hurricane Katrina impacted the coastal city of New Orleans, Louisiana, in 2005. The unprecedented damage, in both lives and dollars, experienced during this natural disaster was partly due to the failure of the levee system designed to protect large portions of the city. As a result, questions as to the methodology, implementation, and adequacy of this flood damage reduction alternative quickly arose. More information about Hurricane Katrina and its impact on New Orleans is provided in Section 12.5.

Bypass channels are used to contain and transport a portion of storm flows away from or around a significantly flood-prone or ecologically sensitive area. These channels are subject to the same limitations as channel improvements to the main stream or floodway. Significant right-of-way and property acquisitions hinder this method's broad applicability. Additionally, there are usually significant excavation costs associated with creating channels where there are no currently existing streambeds.

Nonstructural Methods for Flood Damage Reduction

Nonstructural measures are used to reduce the damage potential of a structure or facility within the floodplain by adjusting the use of flood-prone lands to the flood hazard. This approach contrasts starkly with its structural counterpart, which instead attempts to minimize the flood hazard by actually altering flood flows (Mays, 2001). The amounts of reduction in damage potential achieved by nonstructural measures vary significantly, depending on the mechanism implemented.

There exist a wide variety of nonstructural flood controls, including (USEOP, 1994):

- Flood proofing;
- Flood warning mechanisms;
- Land-use controls such as zoning and development ordinances;
- Flood insurance programs;
- Flood preparedness activities;
- Public awareness and education programs;
- Existing primary and secondary drainage system maintenance;
- Acquisition of floodplain land or voluntary flood-prone property buyouts.

Nonstructural methods have only recently been given serious consideration as a primary means of reducing flood damages, although their potential usefulness has been researched for over 50 years. Alternative options to structural methods were perhaps first articulated by White in 1945. In the 1950s, the Tennessee Valley Authority, with its broad mandate for water-management experimentation, proved an effective pilot program for many nonstructural measures with promising results. Additionally, the nonstructural movement was somewhat popularized by Leopold and Maddock in 1954 as well as by Hoyt and Langbein in 1955, resulting in professional interest and the involvement of the American Society of Civil Engineers in the development of a Guide for the Development of Flood Plain Regulations. However, it was in 1966 that a concerted effort to integrate nonstructural measures into federal flood control policy would take place.

A 1966 Presidential Task Force on Floodplain Regulations, charged with making recommendations for improving the existing federal flood control policy, highlighted five basic objectives for developing a unified national program for managing flood losses. These were:

- Improve the basic knowledge about flood hazards at both the local and regional level;
- Improve planning and coordinating of all new developments in the floodplain;
- Improve technical services available to floodplain managers;
- Create a national program for flood insurance;
- Allow for the future adjustment of federal flood control policy to changing needs.

The Task Force report spurred action to develop uniformity in floodplain management strategies across a myriad of federal agencies, including the Soil Conservation Service and the COE. Additionally, an important result of the report was the creation of the NFIP through the Flood Control Act of 1968.

In 1998, the NWF completed a report, *Higher Ground*, which addressed two of the most important nonstructural issues affecting floodplains—repetitive flood losses under the NFIP and voluntary property buyouts. These issues are addressed in the next two sections.

The National Flood Insurance Program

In response to the failure of the private sector to provide reasonable, cost-effective flood insurance to the average buyer, the NFIP was created by the Flood Insurance Act of 1968 (NWF, 1998). This program was designed to provide cost-effective insurance to property owners who live within the 100-year floodplain in communities that have established a minimum standard of hazard mitigation provisions. The goal, in principle, was simple: Allow the insurance premiums and not the general taxpayer to cover the cost of flood

damages. The required mitigation provisions would enforce floodplain development regulations, guidelines, and limitations. The NFIP was placed under the cognizance of the newly formed FEMA in 1979.

The program has enjoyed some success. Flood insurance premiums covered \$10.4 billion in losses and program expenses between the years 1977 and 1997 (NWF, 1998). As an additional benefit, the program has stimulated additional floodplain management concepts and programs throughout the country.

However, the program has experienced major shortcomings in the form of policy and financial distortions, repetitive loss properties, poor enforcement of regulations pivotal to the success of the program, and significant inaccuracies in floodplain delineations (NWF, 1998). Many opponents of the plan argue that it has actually encouraged development and rebuilding in high-risk areas with the promise of a “limitless guarantee” of governmentally sponsored bailouts.

The existence of repetitive loss properties (RLPs) is a drain upon the efficiency and effectiveness of the program (NWF, 1998). Although comprising only 2% of the properties insured, these properties are responsible for nearly 40% of the program’s payments. The City of Houston is ranked number three on the Top 200 Repetitive Loss Communities Rankings by payments. The city has an estimated 2030 RLPs with total payments in excess of \$114 million (NWF, 1998). The sheer enormity of flood damages in 2004 and 2005 strained the NFIP to an unprecedented level, causing multiple insurance companies to declare bankruptcy and general insurance costs to increase.

Despite this evidence that the NFIP is not the singular flood control answer, its advantages and effectiveness are far reaching. The program has spurred an ever-increasing awareness of the benefits of a sound flood mitigation strategy and even highlighted the environmental benefits of the floodplain when put to its natural use.

Voluntary Buyouts of Flood-Prone Property

The buyout of high-risk floodplain properties from willing sellers and the relocation of at-risk buildings and structures out of the floodplain are non-structural alternatives that have received increased attention in recent years (NWF, 1998). The catastrophic midwest flood of 1993 along the Mississippi and Missouri rivers, as well as several resulting postflood studies and recommendations, spurred national interest in this particular alternative. The essentials of the voluntary buyout program are clearly stated in *Higher Ground* (NWF, 1998).

The overall goal of this alternative is simple to achieve in theory but very difficult to achieve in practice. Theoretically, a one-time expenditure of public funds would result in the evacuation of a specified floodplain area. This cost, although significant, would result in the complete elimination of

flood risk for those areas. Ideally, the long-term financial benefits (in the form of reduced flood losses and an increase in environmental benefits) would more than compensate for the initial cost.

Higher Ground lists the ideal goals of a well-planned voluntary buyout program. These include:

- Seek and secure a combination of federal, state, and local funds for one-time buyouts of high-risk properties from willing sellers;
- Ensure the return of the purchased property to its natural floodplain state;
- Pass legislation that would prohibit the expenditure of any future disaster assistance to that location;
- Provide assistance to former property owners and tenants to move to higher ground and out of harm's way, and, as appropriate, relocate homes and businesses outside the floodplain.

This type of program has specific advantages over structural flood control options as well as many other nonstructural methods. Perhaps its greatest benefit is that it provides permanent disaster relief and help to at-risk people and property. Other strong points include the fact that proper implementation of this plan would end an unlimited disaster relief obligation on public funds by putting tax dollars to the most cost-effective use possible. Lastly, the program would truly realize the environmental benefits of the floodplains by restoring them to their natural ecological functions and thereby provide many of the benefits listed earlier.

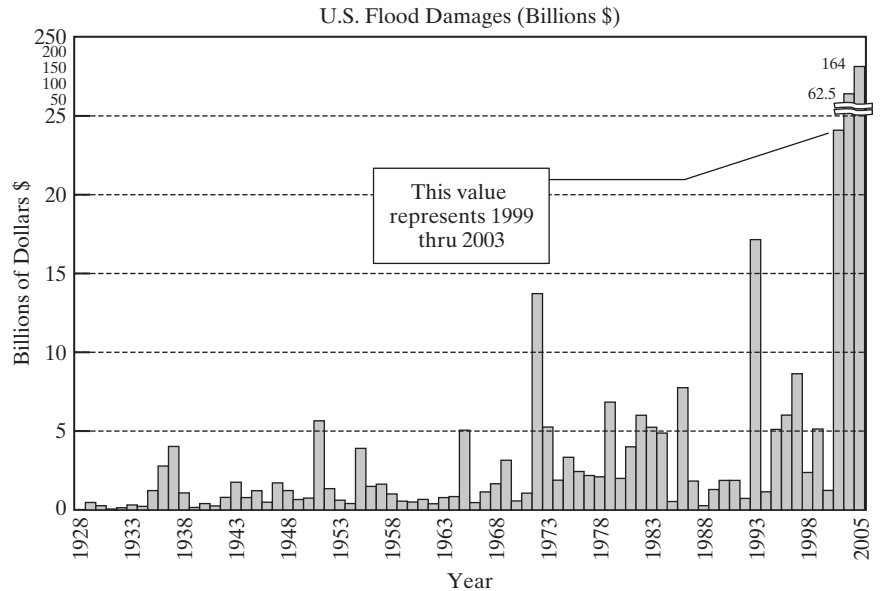
Buyouts are not without limitations and drawbacks. Buyout programs can reduce the local tax base by forcing businesses and significant investors to “look elsewhere”—namely, to other communities. Improper regulatory enforcement or the lack of a sound land management plan after the buyouts may result in renewed development pressure, forcing a change in local regulatory laws that would once again allow floodplain development. Many buyout plans have been criticized as a method of driving low-income residents out of a central, often desirable, part of a community, paving the way for future upscale developments. Lastly, significant costs, time, and careful planning are required to successfully implement a buyout plan, regardless of its scope. The fact that buyouts involve significant changes in land-use patterns, often within well-established areas of a community, may present insurmountable legal, social, and economic difficulties that force the decision-making process toward a more conventional structural method (NWF, 1998).

Despite the expenditure of billions of dollars in structural controls, flood damages throughout the United States have continued to rise. In 1969, near the height of the COE's and Soil Conservation Service's construction of flood control mechanisms, the U.S. Water Resources Council reported

12.4 THE FLOOD CONTROL PARADOX

Figure 12-3

Yearly U.S. flood damages in billions from U.S. Bureau of Economic Analysis, The Why Files & National Weather Service.



nationwide flood damage reduction benefits at about \$1.0 billion annually in response to \$6.1 billion of expenditures toward that effort (A. D. Little, 1973). However, the Council also reported that, based on current trends of land use and development, the total annual flood damage potential for the nation was anticipated to increase from \$1.7 billion in 1966 to \$5.0 billion in 2020 (A. D. Little, 1973). Clearly the 2004 and 2005 hurricane seasons surpassed all flood damage projections as damages skyrocketed into the tens of billions, based on mostly hurricanes in Florida and Hurricane Katrina. Figure 12-3 illustrates the increase in flood loss damages in the United States through the twentieth century into the twenty-first century (NWF, 1998), clearly indicating the seriousness of the problem.

In designing protective works, there are several names for the expected flood a structure should protect against, such as the “design flood or storm,” the “regional flood,” the “standard project flood,” and the “maximum probable flood.” It must be emphasized that one of the most undesirable aspects of structural devices is that they are expected to fail, if an occurrence exceeds their design specifications. The very existence of these protective structures has given rise to an astonishing paradox of increasing flood damages despite the enormous expenditures to reduce them. John Wilkinson, in a report written to the Federal Reserve Bank of Boston in 1967, attempted to summarize factors creating this circumstance.

Memories are short; upstream detention storage and flood walls foster a false sense of security; flood-prone urban land has high locational value; development near the river channel constricts

flood passage, creating backwater over a flood plain on which development also encroaches; developers don't pay for flood relief, rehabilitation or prevention and therefore encounter no financial disincentives to not develop in the expanding floodplain; the next flood produces even greater damage; this in turn justifies more protection; after a brief period, confidence is regained and more development is encouraged. (A. D. Little, 1973).

Fortunately, ways to break this cycle have been understood since before the 1960s. Unfortunately, several technological, political, and economic barriers have hindered their promulgation and acceptance.

Table 12-1 lists some of the deadliest, costliest, and most powerful hurricanes to impact the United States between 1992 and 2008. A quick review reveals that damages quickly escalated during the 2004 and 2005 hurricane seasons, eclipsing the previous damage record of \$26 billion by Hurricane Andrew in 1992. The year 2004 brought the deadly and costly pair of Charley and Ivan in August and September. Their combined damage total of nearly \$30 billion was soon exceeded by Hurricanes Katrina and Rita in 2005. Then in 2008, Hurricane Ike made history as the third costliest storm, causing \$24.9 billion in damage as it made landfall on the Texas coastline and trekked across the central and midwest regions of the country.

The 2005 Atlantic hurricane season was the most active in recorded history, repeatedly breaking many meteorological records. From a period of early June to late December, a record 28 cyclones formed over the Atlantic basin. The 2005 hurricane season produced four category 5 storms and three of the six most intense storms on record. In fact, the year 2005 brought a hurricane with the lowest central pressure in recorded history, Hurricane Wilma, with a minimum central pressure of 882 mb. Explanation of why this particular season was so intense, while important, is best left to textbooks focusing on climatology and larger-scale meteorological factors; however, the interplay between a few of these storms and the rapidly developing coastal United States is of primary concern to hydrologists and a focus of this chapter. This section will briefly highlight two of the most damaging storms to ever hit the Gulf coastline: Hurricanes Katrina and Ike.

12.5 MAJOR GULF HURRICANES: KATRINA AND IKE

Hurricane Katrina

The most damaging storm to ever hit the United States was Hurricane Katrina, which set records as the most costly U.S. natural disaster and the third deadliest hurricane to strike this country. Lashing the southeast Louisiana coast with up to 140-mph winds and a 27-foot storm surge, the storm devastated coastal Mississippi towns, breached the Lake Pontchartrain levees, and flooded 80% of New Orleans. Katrina has proven to be by far

Table 12–1. Selected Major Hurricanes Affecting the United States, 1992–2008

Name	Date	Category	Landfall	Damages	Deaths	Records
Andrew	Aug 1992	4 3	Dade County, FL; Central Louisiana	\$26 billion	23	Second costliest; Fourth most intense at landfall
Alberto	July 1994	Tropical Storm	Florida Panhandle	\$.75 billion	30	
Opal	Sept 1995	3	Pensacola, FL	\$3 billion	9	
Floyd	Sept 1999	2	Cape Fear, NC	\$6 billion	56	
Allison	June 2001	Tropical Storm	Freeport, TX	\$5 billion	41	Most costly and deadly tropical storm
Isabel	Sept 2003	2	Outer Banks, NC	\$3 billion	17	
Charley	Aug 2004	4 1	Fort Myers, FL; McClellanville, SC	\$15 billion	15	
Frances	Sept 2004	2	Stuart, FL	\$9 billion	23	
Ivan	Sept 2004	3	Gulf Shores, AL	\$14 billion	43	
Jeanne	Sept 2004	3	Stuart, FL	\$7 billion	10	
Dennis	July 2005	3	Pensacola, FL	\$2 billion	9	
Katrina	Aug 2005	1 3	North Miami Beach, FL; Buras, LA	\$75 billion	1336	Costliest U.S. hurricane; Third deadliest; Third most intense at landfall
Rita	Sept 2005	3	Texas/Louisiana border	\$10 billion	7	Fourth lowest minimum pressure
Wilma	Oct 2005	3	Naples, FL	\$1 billion	6	Lowest minimum pressure
Ike	Sept 2008	2	Houston, TX	\$24.9 billion	20	Third costliest

the most costly natural disaster on record in the United States, with about \$38.1 billion in insured property losses and \$75 billion total damages.

Katrina began as a tropical depression on August 23, 2005, located 175 miles northeast of Nassau, Bahamas, and by August 25 at 6:30 P.M., Katrina hit the Florida coast near Miami as a category 1 hurricane (Fig. 12–4a). At 10 P.M. on the 27th, the National Weather Service (NWS) issued a hurricane warning indicating the possibility of massive storm surges of 15 to 20 ft from Morgan City to the Alabama/Florida border (Fig. 12–4b). At around 10 A.M. Central Time on August 28, Katrina attained estimated maximum wind speeds of 175 mph. Furthermore, that afternoon Katrina reached its minimum central pressure of 902 mb, the fourth-lowest pressure on record for all Atlantic cyclones at that time. When Katrina finally made landfall near Buras, Louisiana, at 6:10 A.M. Central Time, the maximum wind speeds had reduced to 125 mph and a central pressure of 920 mb, officially making it a category 3 hurricane.

Precipitation associated with Katrina during landfall was greatest just west of the track center. Rainfall total amounts of 8–10 in. were recorded by both rain gage and radar across southeastern Louisiana and southwestern

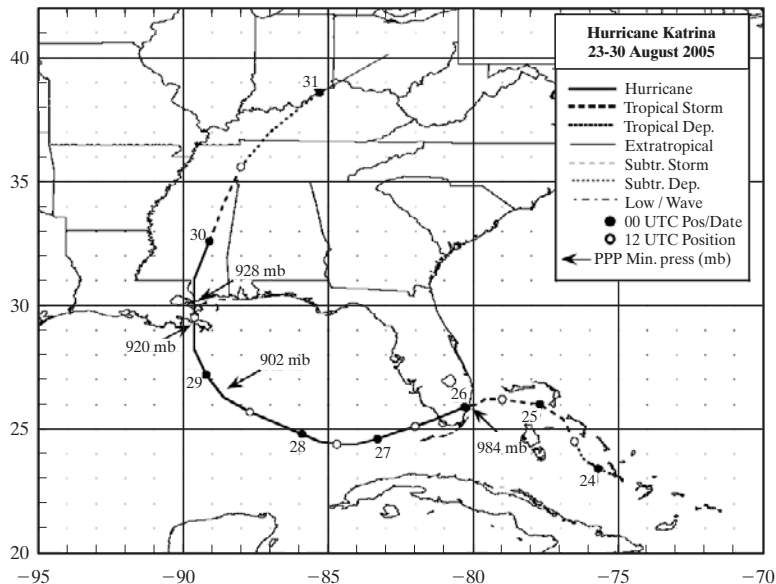


Figure 12-4a
Best track estimates and important minimum central pressures for Hurricane Katrina from August 23 to August 30, 2005. (Source: National Climatic Data Center)

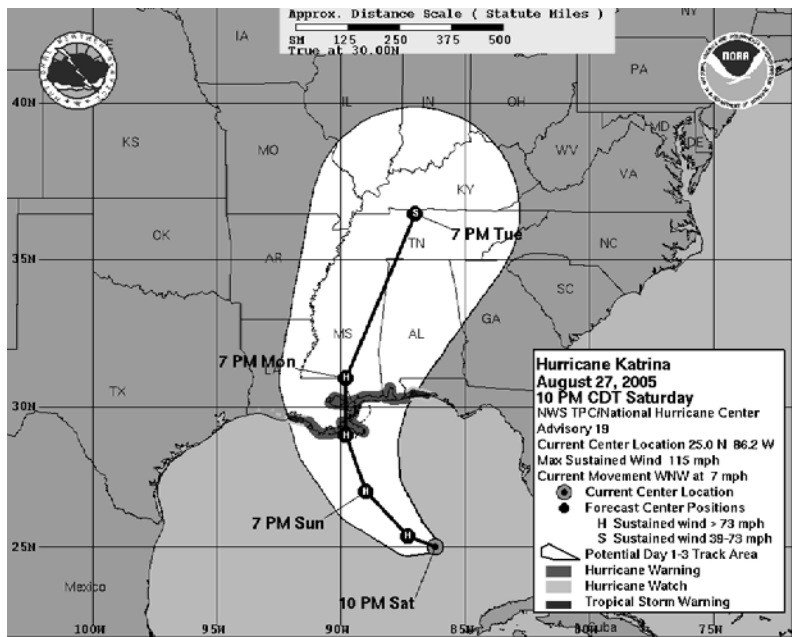


Figure 12-4b
Hurricane Katrina Advisory from National Hurricane Center, 10 P.M., August 27, 2005.

Mississippi. Isolated totals slightly greater than this amount were recorded over eastern Louisiana several miles inland.

While precipitation amounts associated with Hurricane Katrina were not insignificant, the majority of flood-related damages have been attributed to Katrina's formidable storm surge. Katrina's massive size and strength while over the shallow waters of the Gulf enabled its associated storm surge to reach 24–28 ft along a 20-mile-wide swath centered roughly on St. Louis Bay in Mississippi. The complete devastation of buildings and other infrastructure in the immediate vicinity of the storm-surge impact, along with widespread failures of tide gages, makes exact estimates of the storm surge very complicated. It is known, however, that elevated water levels associated with the storm surge reached 6 miles inland in many areas and up to 12 miles inland along bays and rivers.

While the storm surge was highest east of the path of the eye of Katrina, a significant storm surge was also experienced west of the point of land-fall. This resulted in a 14- to 17-foot surge up the Mississippi River as well as the man-made Mississippi River–Gulf Outlet (MRGO). These two waterways converge at a particularly narrow part of the Industrial Canal or Inner Harbor Navigation Canal (IHNC). This “funnel” effect likely intensified the surge, resulting in levee overtopping. At the same time, surge entered Lake Pontchartrain from the east through various passes and, aided by easterly winds, raised the water level of the lake by 10 to 12 ft. The rising waters due to this surge and the failure of levees were the primary causes behind the widespread flooding experienced by New Orleans.

Flood Damage Reduction Strategy for New Orleans: A Pump and Levee System

The city of New Orleans is surrounded by over 14 miles of levees and floodwalls called the Hurricane Protection System, which buffers the city from water-level changes in both Lake Pontchartrain and the Mississippi River. This system protects a low-elevation “bowl” between these two water bodies, reaching as low as 9 ft below the sea level in the residential areas near the lake.

When the surge hit New Orleans, several problems with the levees were revealed. Although the levees were engineered to withstand a category 3 hurricane such as Katrina, the surges that hit the city were more consistent with a category 5 hurricane, as the funneling effect of the MRGO and other existing hydraulic connections drastically increased water velocities and, to a lesser extent, increased the storm-surge heights. More specifically, the Hurricane Protection System was breached in approximately 50 different places. The first 4, and the most devastating, of these 50 breaches were from foundation failures, as the fast-moving storm surge eroded structural support for the levee floodwalls. The remaining 46 breaches were from a direct overtopping of earthen levees and embankments (Fig. 12–4c).



Figure 12-4c
Severe New Orleans flooding due to levee breaks from Hurricane Katrina.

Impacts and the Effectiveness of Flood Damage Reduction Strategies

Katrina was the third deadliest hurricane on record, surpassed only by the Galveston Hurricane of 1900 and the Lake Okeechobee Hurricane of 1928. The official death toll had reached 1836 by May of 2006, with 1296 of those in Louisiana and 238 in Mississippi. More than 250,000 people were displaced by the storm, with evacuees seeking shelter in all 50 states, but primarily in Texas.

The indirect damage estimates from Katrina consist of long-term economic effects that will continue to impact the region for years. The oil industry was most severely impacted, with crippled pipelines, flooded refineries, and damaged, adrift, or capsized oil rigs and platforms reducing the daily oil production by 95% immediately after the storm. The mass population exodus along with millions of lost jobs has severely hurt the local economy and caused a major loss of tax revenue.

The flood control paradox discussed earlier is very relevant in this case study. At the time of publication of this text, the question “Should we rebuild?” will have been answered, and the rebuilding of New Orleans will likely continue into the future. Changes are in place that many hope will reduce the damage susceptibility of this great city, including improvements to the pump and levee system, improvements in the skill of predicting hurricane storm track and intensity, and improved storm-surge modeling. One example of these upgrades (completed in 2010) is the IHNC Lake Borgne Surge Barrier, a 1.5-mile-long blockade that crosses the confluence of the Gulf Intracoastal Waterway and MRGO. However, though such precautions are being taken, only time will tell if the decision to develop in the Mississippi

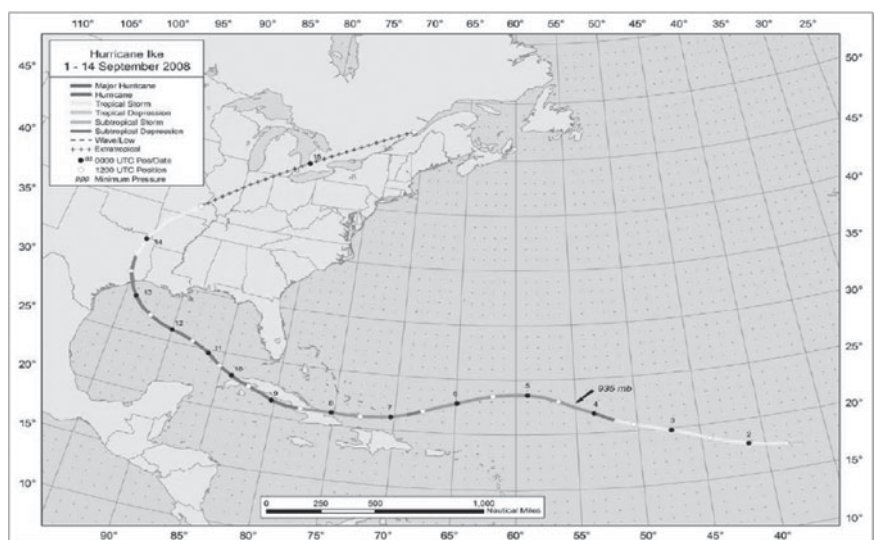
floodplain along the Gulf Coast will again prove to be very costly in terms of life and property. For more information, consult the National Hurricane Center Tropical Cyclone Report on Hurricane Katrina (Berg, 2009).

Hurricane Ike

The year 2008 shocked the Texas coastline with the most destructive storm to hit the state since the Galveston Hurricane of 1900. The third costliest storm in the U.S. history, Hurricane Ike brought \$24 billion in damages and over 20 deaths, thereby reemphasizing our nation's vulnerability to natural disasters.

On August 28, 2008, a tropical wave formation, and what would become Hurricane Ike, moved off the coast of West Africa. The wave developed into a hurricane over the Atlantic Ocean (Fig. 12–5), and after reaching its peak intensity as a category 4 storm of 145 mph and 935 mb on September 4, Hurricane Ike made landfall at the Bahamas on September 7. On September 8 and 9 the storm made landfall twice in Cuba, and by September 11 Hurricane Ike was in the Gulf of Mexico as an unusually large category 2 hurricane with a wind field 450 miles in diameter (Fig. 12–5). Hurricane Ike made landfall at Galveston, Texas, as a category 2 storm on September 13 at 2:10 A.M. CDT with winds of 110 mph and a central pressure of 950 mb (Bedient et al., 2011). The damaging storm surge during Ike, which may have been worse than expected due to the storm's abnormally large wind field, motivated the NWS to enhance its analysis and prediction of storm surge and to develop effective ways of communicating the associated risks to the public. The National Hurricane Center's probabilistic storm surge product, which became operational in 2009, is one example of this effort (NHC, 2010).

Figure 12–5
National
Hurricane Center
track for Hurricane
Ike.



Although the storm did bring rainfall to the Texas coastline, Hurricane Ike was first and foremost a major storm surge event. Bolivar Peninsula near Galveston Island received a storm surge of 17.8 ft, and the highest water-mark collected by FEMA was 17.5 ft in Chambers County, which is located approximately 10 n mi inland. Early waves from the storm were already impacting the coast 24 hrs ahead of landfall, and by the evening of September 12 the surge was close to 10 ft on Galveston Island and 15–20 ft on Bolivar Peninsula. Since the Gulf side of Galveston Island is protected by a 17-ft seawall, flooding from the storm surge on the island began on the bay side (Bedient et al., 2012). Bolivar Peninsula, however, which is not protected, was victim to devastating storm surge that cleared entire beaches and left only piles of debris across much of the peninsula (Fig. 12–6).

The damage and destruction that Hurricane Ike left in its wake has highlighted the importance to prepare for and to predict the potential impact



Figure 12–6

A single home left standing on a beach in Gilchrist, Texas, on Bolivar Peninsula after Hurricane Ike. (Photo courtesy Bryan Carlile.)

from future storms for the Texas coastline and for the entire nation. According to Bedient et al.'s *Lessons from Hurricane Ike* (2012), special attentions are being given to coastal flood insurance, structural and nonstructural approaches to flood mitigation, and future development. The Houston Ship Channel, for example, is of particular concern because of its high risk to storm surge and the national disaster that would ensue should it be compromised. In an effort to evaluate similar vulnerabilities along the Gulf Coast, research is being conducted at the University of Texas as part of the Severe Storm Prediction, Education, and Evacuation from Disasters (SSPEED) Center to simulate storm surge and its consequent impacts from various storm scenarios. These methods have already been used to accurately model the Hurricane Ike surge prior to and after the storm (Dawson, 2011). For more information, interested readers may visit the SSPEED website at <http://sspeed.rice.edu>.

12.6
IMPROVED
STRATEGIES
TOWARD FLOOD
MANAGEMENT

The continual increase in flood-related damages, whether resulting from tropical storm activity or severe thunderstorms associated with more frequent frontal activity, illustrates the need to improve our flood management strategies at all geographic scales, from local to national. Improvements in weather forecasting skill and local efforts that rely on structural methods of flood protection will continue to play a vital role in reducing flood damages; however, the increasing population along coastal areas and in larger, urban centers demands a greater reliance on nonstructural methods in both short- and long-term flood management strategies. Many cities across the nation are participating in Low Impact Development (LID) practices to control runoff volume and pollutant loading. More information on the many different approaches to LID is summarized in the next section. A different, but equally impactful nonstructural approach that has worked successfully in Houston, Texas, is the Rice University/Texas Medical Center Flood Alert System (Rice/TMC FAS). The Flood Alert/Warnings Systems sections will discuss the role that flood alert systems can play in improving flood management strategies, particularly in quickly responding urbanized watersheds. Many other radar-based systems are in operation around the United States and abroad.

Low Impact Development (LID)

The utilization of LID designs is meant to decrease the adverse hydrologic effects of construction and urbanization on a given watershed and to allow the given area to best mimic predevelopment hydrology. LID has recently become very popular for managing urban runoff, and there are many different categories of practices, including but not limited to conservation designs, structural and landscape designs to increase infiltration, filtration practices to treat runoff, and runoff conveyance practices.

Two common LID methods that contribute not only to decreasing rainfall runoff but also to filtering rainfall are known as pervious pavement and vegetative swales. Both work off of the same principle: replacing impervious cover, such as concrete, with pervious surfaces that allow for rain to percolate through a soil medium acting as a filter and flow out of the area at a slower rate. Pervious pavement replaces traditional impervious concrete or asphalt with a pervious concrete paver that is placed on top of layers of other materials, which facilitate the movement of rainfall to a series of drains placed under the pavement that transport the percolating water to a desired location (ICPI, 2010). Vegetative swales work in a similar manner, with the main difference that the swales are self-contained planters that typically receive runoff directly from roofs or an impervious surface such as the road. The runoff that is collected then percolates through the soil medium found in the vegetative swale and is discharged at a desired location (PBES, 2011).

Another LID practice that can be implemented is a rainwater catchment system. These systems collect water from a catchment surface, such as a roof, through a series of gutters, and the water is piped into a large cistern for storage. This technique aids not only in decreasing the effects of urbanization on runoff, but also in arid and drought-stricken areas; it can be used to harvest rain water that can be used for drinking water (Mays, 2011).

In addition to on-site green infrastructure, conservation designs are built on the scale of a neighborhood to preserve open space in order to increase either infiltration or evapotranspiration. Higher-density development is one method that attempts to accomplish this goal by grouping populations in a smaller space, such as a high-rise apartment building, in order to vacate other areas that may be used for parks and other types of open space. For instance, 4 single-family homes in suburbia may occupy one acre of land, whereas apartments may take up the same amount of space and house 10 or more families. Additionally, because the soil on a single-family plot of land is compacted by heavy machinery, lawns often behave like impervious surfaces (USEPA, 2006). In fact, the U.S. Department of Agriculture (USDA) has shown that larger lawns do not compensate for the increased runoff that is created when a watershed is developed (USDA, 2001). Instead, utilizing open space for pocket wetlands, forestry, and parks can help attenuate flow and reduce the volume of runoff in a watershed. The San Antonio Water System is a good example of how legislation can be used to control stormwater runoff and increase the water resources in a community. The organization has designated Floodplain Preservation Areas, which are located within the Edwards Aquifer Recharge Zone, to limit the amount of activity that is allowed. Furthermore, Floodplain Buffer Zones run parallel to the Floodplain Preservation Areas and only contain pervious cover (SAWS, 1995). More information on water resources in San Antonio can be found in Chapter 13.

LID can have a powerful effect when a number of different techniques are combined together in a designated area such as a planned community. The Woodlands is a 37-yr-old totally planned community north of Houston,

Texas, that, when originally designed in the 1974, used many techniques that were precursors to the LID designs we see today. The master drainage plan of the development was planned from the beginning to minimize any adverse impacts on the 100-yr floodplain. The development criterion was to minimize any change in the existing, undeveloped floodplain at two central points. More than one-third of The Woodlands was designated as open space, and is either found in its natural state or maintained for recreational use, parklands, or other community facilities. Also, to minimize the impact of urbanization on the peak runoff response, the drainage system was maintained in its natural state as much as possible to provide higher storage capacity and resistance to flow within the channels. Drainage into the major water courses is provided by numerous broad, shallow swales with slopes generally less than 1%, and they are covered in native vegetation to impede stormwater flow (Bedient and Huber, 2002).

Flood Alert/Warning Systems

Flood alert/warning systems are becoming more prevalent as tools to reduce the flood damage susceptibility of critical areas in watersheds throughout the United States. These systems are designed to collect, handle, analyze, and disseminate hydrologic information, ideally in real time, for the purpose of providing accurate advance warnings of an impending flood condition. Their incorporation in existing and newly developed flood management strategies should be encouraged. An increase in the use of these types of systems is likely, given the following factors: the continued growth of computer-mediated voice and data communication systems, including the Internet and data-capable wireless devices; the increased availability of high-quality hydrologic data, including radar rainfall, LIDAR elevation datasets, and GIS-formatted hydrography; and the overall resurgence of the nonstructural flood control method as a tool to complement flood damage reduction strategies.

Recent technological advances in hydrology and in a number of other fields, such as meteorology and computer science, have made available a variety of tools capable of increasing the performance of a flood alert system. This is particularly true for alert systems designed for quickly responding urban watersheds. Some of these advances include the continued research in the application of Next Generation Radar (NEXRAD), the development of more accurate short-term Quantitative Precipitation Forecast algorithms, the development of powerful yet efficient distributed parameter and lumped parameter hydrologic models that run in real time, and the linkage of all of these advances within the ever-improving GIS field (Bedient et al., 2003; Fang et al., 2008).

The above-listed factors have permitted these types of systems to successfully issue flood warnings at specific locations with increasing lead time at even the small, urban watershed scale. Once installed in these

watersheds, and accepted as part of a flood management system, time-critical actions that may result in significant costs to both public and private organizations can be taken only when necessary and with ample time to minimize flood damages.

Traditionally, warnings provided by the NWS for watersheds on the order of 100 square miles or less are issued as blanket statements for broad areas such as a county or quadrant of a county. These warnings are issued quite liberally with the aim of “failing safe”; however, the downside to this approach is the frequent false-alarm ratio that results, making the public skeptical and hesitant to take action in response to future warnings. Furthermore, where small-scale warning systems do exist as part of an ALERT (Automated Local Evaluation in Real Time) System, these systems are typically rain gage based. While these gage-based flood alert systems have fulfilled the role of providing flood notification to many people and have undoubtedly saved millions in both lives and property, they fail to take advantage of many of the above-listed technologies to improve lead times and system accuracy.

The next section provides a general overview of the development and operation of both the original RICE/TMC FAS and the second-generation system known as FAS-2. FAS-3 was released in 2010 with marked improvement and incorporation of Google Earth mapping features (Fang et al., 2011). Figure 12–7 shows the location of Brays Bayou and the Texas Medical Center. FAS-3 represents a fully operational system that is currently in use for flood warning. While this system was created for an urban watershed,

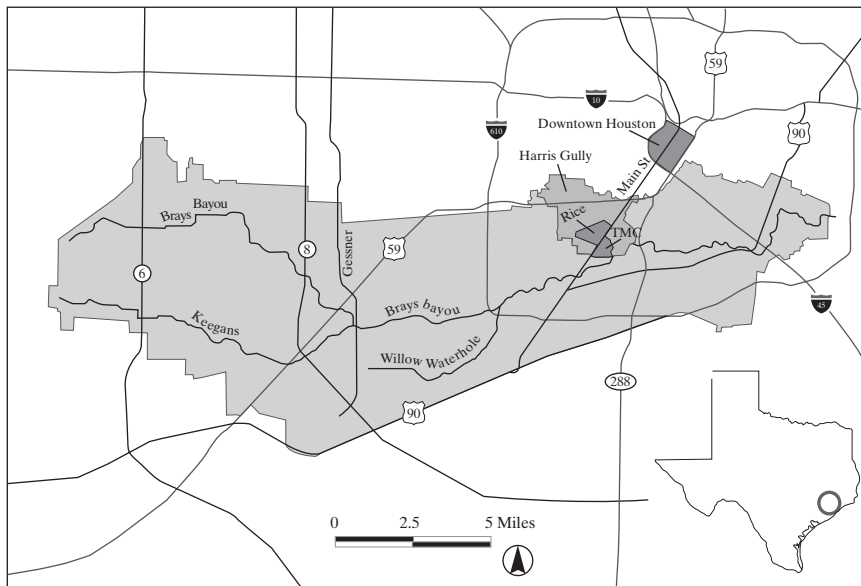
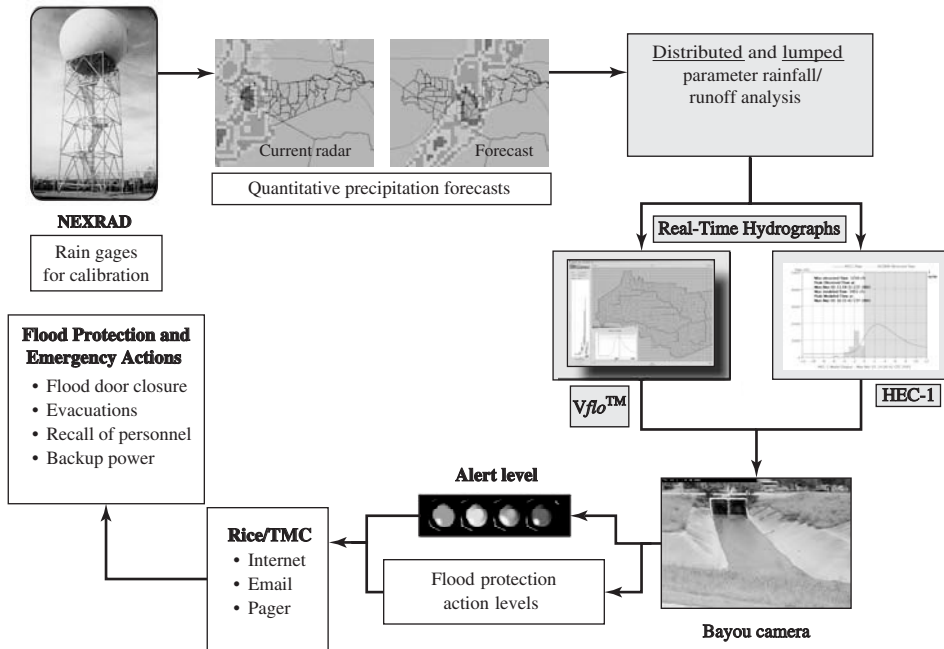


Figure 12–7
Map of Brays
Bayou & Texas
Medical Center.

**Figure 12-8**

Schematic representation of data processing steps and information flow in the FAS2 system. (Source: <http://fas2.rice.edu/>.)

the methodology and technology should be applicable across a broad spectrum of watershed types, provided that the need for more timely and accurate flood forecasts exists—a condition that almost always prevails. Figure 12-8 presents a flowchart representation of data processing steps and information flow in the FAS-3.

The Rice/Texas Medical Center Flood Alert System

The original FAS was an integrated system that gathered NEXRAD data from the NWS and extracted rainfall totals over the Brays Bayou watershed in southwest Houston, Texas (Bedient et al., 2000; Vieux and Bedient, 1998). It used this information to provide flood forecasts for a single location in the bayou near the Texas Medical Center—the world's largest medical complex, consisting of numerous hospitals, hotels, restaurants, public transportation systems, and medical schools. The area is prone to flooding, owing to its location in the downstream portion of a nearly completely urbanized watershed called Brays Bayou. This bayou and the TMC have been impacted by two major floods and several minor floods over the past 25 years.

Most urban alert systems operated by local emergency management agencies still rely solely on an often limited number of rain gages and stream

gages to provide notifications. For example, the Harris County Office of Emergency Management ALERT system still maintains a strict reliance on rain gage/stream gage for dissemination of warnings. While its network is perhaps one of the densest in the world, with approximately 1 rain gage per 10 square miles, the system remains a classic reactive vs. proactive alert system—waiting until at least 2 in. of rainfall is detected by a rain gage and a corresponding stream flow gage nearby has verified increasing flow in the appropriate watershed prior to issuing an alert. This type of reactive approach, while almost never issuing a false alarm, is capable of providing only about 30 minutes or less of lead time (Bedient et al., 2003).

The Second-Generation Flood Alert System (FAS2)

The second generation of the Rice/Texas Medical Center Flood Alert System, known as FAS2, has been developed with the aim of addressing many of the shortfalls of the original system. This process has focused on utilizing many advances in the fields of radar sensing, real-time hydrologic modeling, and improved information transfer via digital systems such as the Internet, PDAs (personal digital assistants), and cell phones. Four general areas were identified where the existing system could be improved: lead time, accuracy, effectiveness, and reliability.

Color-coded flood alert action levels were created for various flow levels predicted by the FAS for Brays Bayou at Main Street. The alert status was based on nomograph-predicted flows. A flood caution (yellow) would be issued for predicted flows between 20,000 ft³/s and 24,000 ft³/s. A heightened alert status (orange) would be issued for predicted flows between 24,000 ft³/s and 28,000 ft³/s, when flooding would be considered possible, given additional rainfall over the watershed. A final alert status (red) would be issued for predicted flows greater than 28,000 ft³/s, signifying that severe flooding at the TMC would be probable, given the combination of existing flow in the Bayou and current rainfall estimates.

The animated radar rainfall images were available at various time periods and scales. For example, end users could request the last 24 hrs of radar rainfall over the entire southeast Texas area and then select the last 3 hrs of detailed data specifically over Brays Bayou. These images were routinely used to analyze historical storm movement as well as to obtain a qualitative feel for the growth and decay trends for individual storm cells.

The FAS's two cameras, called BayouCams, enabled constant visual monitoring and verification of flow levels at vital confluences. Prior to their installation, TMC security personnel were dispatched to physically observe these levels. The images from the BayouCams were archived during significant storm events, providing valuable data for post event analysis and system performance evaluation.

Improving the accuracy of the system was accomplished by a variety of means, including improved radar data input and resolution, real-time

calibration of the radar with rain gages when available, and the development of real-time hydrologic models—both distributed and lumped parameter. A real-time interface for the industry-standard HEC-1 hydrologic model was developed, permitting the hydrologic predictions developed by these models to take greater advantage of the spatial and temporal distribution of real-time radar data. This real-time hydrologic model also provides significant lead-time improvements by providing estimates of when peak flows would actually occur.

Information dissemination tools besides the Internet were added to the new system, including automated notification via cellular phones, pagers, and e-mail. New alert action levels were developed to incorporate the increased lead time and accuracy from the new system. This had the desired effect of reducing the number of false alarms to the TMC. Lastly, the system was tested from start to finish over a number of actual storm events. The system has worked very well for 10 major storms since 2001 with an observed vs. predicted r^2 value of 0.90. FAS2 has generally predicted floods with average differences of 0.88 hr for time of peak and 3.6% in peak flows for the overall performance. System reliability was also improved by establishing multiple server sites at various locations, including Rice University, TMC, and Norman, Oklahoma (Fang et al., 2008).

The Third-Generation Flood Alert System (FAS3)

The advanced radar-based flood alert system (FAS) has evolved through two generations using available technologies and enhancements and has played an important role in flood warning and flood control aspects for the Texas Medical Center since it debuted. After years of urbanization in the watershed, the hydrologic model as a core engine in the system has been updated with recent routing information and loss rate factors to provide more reliable prediction information.

The system has been recently upgraded into its third generation (fas3.flood-alert.org) by incorporating a hydraulic prediction tool—Floodplain Map Library (FPML) so that mapped warning information in Geographic Information Systems (GIS) and Google Maps can be provided to emergency personnel in real time. The FAS3 system employs Asynchronous Javascript and XML (AJAX) technology to reduce web traffic while allowing for more timely display of updates. The web browser can periodically check with the server to see if an update has occurred. In this way, more frequent checks can be performed for shorter durations, which is very important for end users under severe weather conditions. In addition, with the popularity of Google Maps—a web mapping service application and technology provided by Google—the FAS3 system displays the FPML within the Google Maps frame (FAS3.rice.edu). When rainfall occurs in the watershed, the FAS3 algorithm determines a floodplain according to a rainfall intensity estimate that is computed over the last 3 hrs of radar rainfall

data. Rainfall estimates are updated whenever the FAS3 server receives new data. Showing colored floodplain mapping information in Google Maps enables emergency personnel to better understand weather conditions during any event (Fang et al., 2011).

Chapter 12 introduced essential information concerning floodplain management issues, flood control alternatives, and flood control policy. The U.S. Army Corps of Engineers (COE), the primary agency in charge of these issues at the national level, was introduced, and several aspects of the agency were discussed. Specifically, the chapter focused on recent changes in flood control practice, which are now being adopted by the COE. These changes, often in the form of encompassing more non-structural flood control measures, have enabled a broader and more environmentally conscious approach to floodplain management.

SUMMARY

Specific examples of both structural and nonstructural flood control measures have been presented for the reader to more fully appreciate the advantages and disadvantages of each when used in any floodplain management scheme. A heavy emphasis was placed on the environmental impacts associated with channelization. Additionally, floodplain property buyouts and the NFIP were presented as two viable nonstructural flood control alternatives to traditional, structural approaches. The role of the FEMA in flood disasters was briefly discussed.

The chapter presents a detailed treatment of Hurricanes Katrina and Ike and their enormous impact along the Gulf coast. In particular, Katrina created the greatest flood damage (over \$75 billion at time of printing) ever seen in a major metropolitan area (surpassing T. S. Allison in Houston in June, 2001). Hurricane Ike, the first major direct hit on Houston since 1983, damaged more than 2500 homes and displaced large numbers of people, while generating the third highest damage in history (\$24.9 billion) (Bedient et al., 2012). These storms demonstrate the power and fury of Mother Nature in spite of the millions that have been spent over the years on floodplain management and levee construction, especially in New Orleans.

The chapter concludes with a section on improved strategies for flood management, including low impact development methods and flood warning systems. These methods hold great promise for the future to help promote sustainable watersheds.

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Chapter 13

Case Studies in Hydrologic Engineering: Water Resources Projects

—Antonia Sebastian



The Hoover Dam Bypass Bridge, completed in 2010, spans the canyon nearly 900 feet above the Colorado River.



A view of the massive Hoover Dam Structure was constructed between 1931 and 1936 across Black Canyon.

13.1 INTRODUCTION

The next generation of modern hydrologists will face unprecedented challenges in dealing with the evaluation and investment in existing water resource projects as well as the development of new and innovative ones. This chapter explores some world-renowned water resources projects that demonstrate hydrologic applications at the highest level of achievement. These include projects for the water systems in central Texas, the Colorado River basin, and international projects on the Thames River near London and in China and Southeast Asia.

Central Texas is a prime example of a region facing issues at both ends of the spectrum. The complex ground water aquifer system has very little long-term storage capacity. Drought, coupled with ever-increasing population growth in the region, has forced water managers to address conservation and look toward alternative water supply sources to maintain a sustainable system. This region is also a part of flash flood alley, an area where the highest rainfall and runoff rates in the country contribute to dangerous flooding. Major water projects have been developed to respond to these challenges.

The City of San Antonio, home to 1.3 million people, lies in the midst of central Texas. The San Antonio River, running directly through the city's

downtown, is one of the premier examples of structural flood control used to expand economic opportunity. Levees, dams, tunnels, and sluice gates have all been built to control the river and to protect downtown. The San Antonio Riverwalk and expanded river reach are examples of sustainable water development projects that have become the center of a robust tourist economy and the livelihood of the city.

Water issues in the western United States far exceed those of central Texas and are some of the most complex in the world. Over 20 million people rely on the Colorado River for drinking water and irrigation, and much of the U.S. food supply is produced from farms that use this water. However, this unruly source has only recently been tamed. The seven western states that rely on the Colorado River came to an agreement in the 1920s over the division of the Colorado River's water, but today the annual flow of the river is over allocated. The Colorado River is one of the most heavily dammed, regulated, and litigated rivers in the United States and serves as a prime example of how important water resources are to the fabric of life and culture.

The greatest and most successful dam project on the river was the Hoover Dam built in the 1930s. Not long after, Glen Canyon Dam was built to guarantee water to the Upper Basin States. Hoover Dam has been a great success for many reasons, but Glen Canyon Dam has proven to be more of an environmental disaster. Not only has it greatly affected flows and wildlife in the Grand Canyon National Park, but its reservoir has inundated one of the most awe-inspiring canyons on the Colorado River. Internationally, dams have led to heated discussions and often their negative effects can outweigh the benefits of construction.

The Thames River in England is an example of a coastal river system that faces both water quality issues and flooding problems. Historically, the Thames River has been contaminated with sewage effluent from London for hundreds of years. More recently, a modern sanitary sewer system has been designed that carries sewage away from the river. Additionally, the Thames River is vulnerable to severe flooding from upstream snowmelt and rainfall, as well as tidal movements and storm surge. Engineers have installed and designed a storm surge barrier to protect London's infrastructure and population.

China is a major developing economy that has enormous water supply, flooding, and water quality problems. Many of its rivers have been or are being dammed for flood control in many regions, and already there have been serious environmental and sediment issues. The recent opening of Three Gorges Dam in 1994, the largest in the world, is not without a host of problems. The Chinese government also admitted the dam has negatively impacted downstream river water, sediment transport, and migration of citizens (Kent, 2011).

The above water resources projects provoke a series of important questions that puzzle modern engineers: To what extent are we able to tame and reroute rivers to serve our needs? How high and how big must we build before we become a danger to ourselves? At what point have we consumed all the renewable resources in our aquifers? How far must we

regulate and litigate to protect our communities and provide water for future generations? And finally, what role will climate change play in future water resource projects?

13.2
THE CITY OF SAN
ANTONIO—DEEP
IN THE HEART OF
TEXAS

The City of San Antonio is the seventh largest city in the United States. It is home to 1.3 million people and is the largest city to rely solely on groundwater to sustain its population. The unique early history of the city sets the stage for the development of an economic metropolis located directly on the San Antonio River. Although the city receives approximately 29 inches of rain annually, rainfall can vary anywhere from 10 to 52 inches in a given year. In some years, riverbeds run practically dry causing citizens to worry about the state of the aquifer and triggering usage restrictions. However, in other years the river flows out of banks wreaking havoc and destruction.

Water managers and policy makers in the City of San Antonio have been forced to address conservation, flood abatement, and sustainable urban planning in a region that relies heavily on a fractured limestone aquifer for its major water supply and a river prone to flooding for its economic vitality. San Antonio's proven success in conserving and managing water has made it a national leader in water supply management. For example, the Riverwalk and expanded river reach is one of the most successful water resources projects in the United States and has motivated similar development internationally.

A Brief History of Water Use

Not long after the Spanish Conquistadores began arriving in Texas in 1691, they discovered that the area around what is today San Antonio could be a potential colonial frontier north of Mexico and a permanent stronghold against French expansion from Louisiana. By the early 1700s, the Spanish had built five missions along the banks of the San Antonio River, which became the largest concentration of Catholic missions in North America (Fig. 13–1).

The missions acted as centers for military, religious, and cultural instruction of the native Indians, and the ranching practices developed by the Spaniards at the missions later became the basis for the U.S. agriculture and cattle industries. One agricultural technique used by the Spaniards was to transport water using a series of irrigation ditches, dams, and aqueducts known as acequias (Fig 13–2). Seven acequia systems linked the five missions creating a 15-mile network that was capable of irrigating 3500 acres. This was the first municipal water distribution system in the lower 48 states and continued to serve as the city's sole water system for almost two hundred years (Eckhardt, 2011). Even today, parts of the system are still in use.

San Antonio is best known for the historic Battle of the Alamo, where a few dozen defenders held their ground while Mexican soldiers laid siege to the mission. In the end, Texas gained its independence from Mexico at San Jacinto near Houston in 1836, and was annexed into the United States in



Figure 13-1
Mission San Antonio de Valero, built in 1718 along the San Antonio River, is known today as the Alamo.

1845. After the Civil War in 1865, Texas longhorn cattle became prized for their hardy nature, and over the two decades immediately following the War, cattle drives were a common site throughout the state. Cowboys were able to lead the cattle north through Texas, Oklahoma, and Kansas for over 20 years, due to ample water supplies along the trails. The last big drive was in 1885 when a severe winter followed by drought forced many to sell their animals.



Figure 13-2
The Espada acequia over Piedras Creek was built by early Spanish settlers to carry water from the San Antonio River to the missions for irrigation and drinking water.

In early 1901, oil was discovered in Texas, and the East Texas Oil Field turned out to be massive and oil wells sprang up overnight. Further investigation revealed that much of the state sat on oil reserves, and the race for the black gold began, an industry very dependent on a rich source of water.

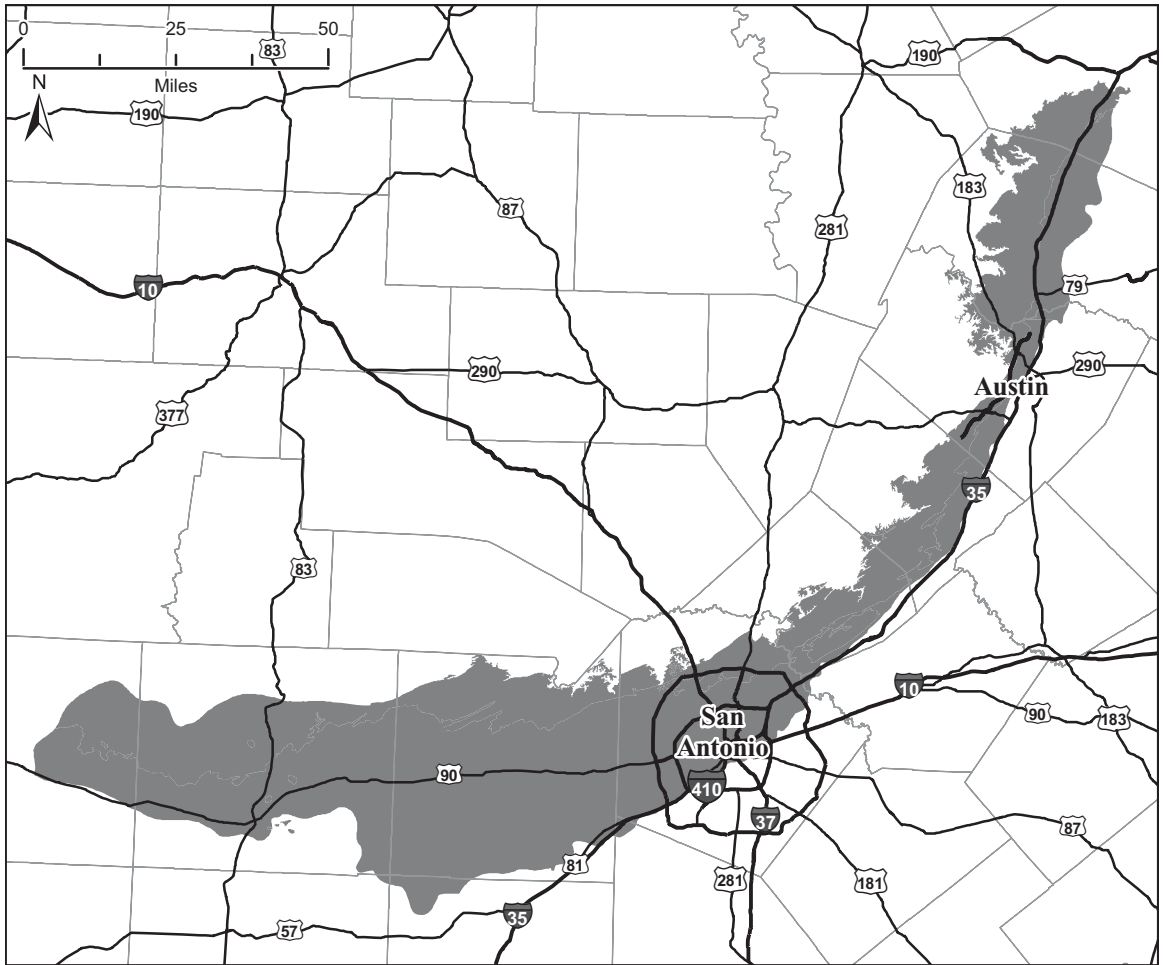
In the 1890s, a prominent figure in San Antonio, George Brackenridge, observed that spring flows were fluctuating wildly corresponding to the amount of rainfall. He was concerned that the springs were not a sustainable source of water for the growing population. In response, he drilled major artesian wells on his property near the headwaters of the San Antonio River, and other citizens began drilling wells due to a drought that greatly reduced spring flows. The abundant underground resource and the improvement in well drilling methods at the turn of the century allowed the expansion of drilling to take place and created the platform for substantial urban growth near San Antonio.

The Edwards Aquifer

The Edwards Aquifer is the major aquifer serving central Texas and San Antonio. About 200 million years ago, central Texas was covered by a shallow sea. As this sea oscillated back and forth, the Edwards limestone formation was exposed to the elements. During the late Cretaceous Period, sediments were deposited on top of the exposed rock, trapping the water in the crevices between the old and new limestone and forming the Edwards Aquifer. About 70 million years ago, wind deposited even more sediments on top of the Edwards formation, and the weight of the sediments caused faults to form between the Edwards Plateau and the Gulf of Mexico. The most visible expression of these faults is the Balcones Escarpment which marks the transition of the Gulf Coast Prairie to the Texas Hill Country.

Some of the most extreme rainfall-runoff rates in the country occur along the Balcones Escarpment. As streams flow across the permeable, fractured carbonate rocks, their base flow seeps through the rocky, clay layer and into the conduits of the recharge zone, an area of approximately 1250 square miles (EAA, 2011). Beneath the surface of the recharge zone, water flows into the artesian zone of the aquifer. Here, the water is under so much pressure between limestone confining units that it can reach the surface without pumping, in the form of artesian wells or springs. In parts of the aquifer these springs seldom go dry, while in others their flow varies greatly from year to year. Average groundwater velocities can range from 27 feet/day to 6000 feet/day depending on the region of the aquifer (Ardis and Fahlquist, 2004).

Today, the Edwards Aquifer lies beneath eight counties, including the more heavily populated Bexar, Comal, Hays, Uvalde, and Medina Counties (Fig. 13–3). It covers an area of approximately 3600 square miles and is 180 miles long and anywhere from 5 to 40 miles wide (SAWS, 2011).

**Figure 13–3**

The Edwards Aquifer serves approximately 2 million people in Central Texas.

The demographics of the region above the aquifer include farm and ranchland, urban metropolises, and industrial sectors, as well as tourist and recreational areas along the rivers and springs. The aquifer also supplies water to major Texas cities, including San Antonio, serving an estimated 2 million people in 2010 (EAA, 2011). Annually the aquifer is expected to yield approximately 0.45 million ac-ft of water (Donahue and Klaver, 2005). Aquifer water quality is generally good, but there are some issues of concern.

The aquifer's capacity has been estimated to be about 173 million ac-ft of water. It is considered one of the most productive groundwater

systems in the United States (EAA, 2011). However, because the aquifer is artesian, water levels are highly dependent on the amount of annual rainfall. In years with heavy rainfall, the water flows freely from springs, but during long droughts, the activity in the aquifer decreases and the economic and environmental costs of pumping water from the aquifer increase significantly.

Alternative Sources of Water

The Edwards Aquifer Authority (EAA) projects that the population served by the aquifer will grow from 1.9 million in 2010 to 3 million by 2050 and that annual demand will reach 0.5 million ac-ft. Such projections have led to serious discussions over the future availability of the water resource and the need for long-term resource planning, including the diversification of water resources. The San Antonio Water System (SAWS) has led the charge, developing a 50-year water plan that identifies ways to create long-term water storage, employ recycling and conservation goals, and develop alternative water sources.

One of the most groundbreaking water resource developments was the creation of the Twin Oaks Aquifer Storage and Recovery (ASR) facility by SAWS. Completed in 2004, at a total cost of \$255 million, the facility is located in the Carrizo Aquifer. Water can be injected into the sand aquifer, creating a bubble or dome that can be accessed later when there is a shortage of water in the Edwards Aquifer. During wet years, water is pumped from the Edwards Aquifer into the Carrizo Aquifer and stored at the ASR facility. Then, during drought periods, the water is returned to the distribution system to meet demand. Unlike a surface water reservoir, no water evaporates. Estimates are that more than 0.1 million ac-ft of water can be stored at the facility (SAWS, 2011).

The city has been using recycled water to cool electrical generating plants since 1963, and by the year 2000 San Antonio had the nation's largest recycled water distribution system (Eckhardt, 2011). More than 80 miles of pipeline can provide about 20% of the city's total water capacity. Recycled water is supplied to commercial and industrial customers for non-potable uses, such as irrigation and cooling, and used to supplement base-flow in creeks and streams as well as the San Antonio Riverwalk. SAWS has taken the initiative to educate the public about the uses of recycled water, encouraging them to conserve and heightening awareness about water shortage issues in the area. This effort has been complimented by the development of more efficient home appliances and landscaping. Since 1987, despite a 50% increase in water users, there has been no increase in water consumption (SAWS, 2011).

Despite conservation initiatives, the Edwards Aquifer does not have the capacity to solely supply the city's water in the future. SAWS has looked to the Lower Colorado River Authority (LCRA) in Texas to supplement

water supply by piping water from the Lower Colorado River to San Antonio. The equivalent of 50% of annual withdrawals from the Edwards Aquifer could be drawn across the pipeline (Earl and Votteler, 2005). In 2001, SAWS and LCRA reached an agreement, allowing San Antonio to purchase 150,000 ac-ft of water from LCRA for up to 80 years. The project was estimated to cost about \$1 billion, but by 2008 the estimated cost of the project had increased by another \$1 billion and the maximum projected yield decreased to 90,000 ac-ft. Then in 2009, the LCRA predicted that regional population growth made supplying additional water to San Antonio unfeasible. By May, SAWS declared that the LCRA was in breach of contract and ended all discussions.

SAWS has also proposed that by 2060 one alternative may be to desalinate ocean water and pipe it back to San Antonio. However, the economic and environmental costs are enormous and the feasibility of such a project would need to be further explored. Many other aquifer and piping projects and proposals exist, but will not be covered here. For more information, please refer to the website edwardsaquifer.net.

Edwards Aquifer Management and Protection

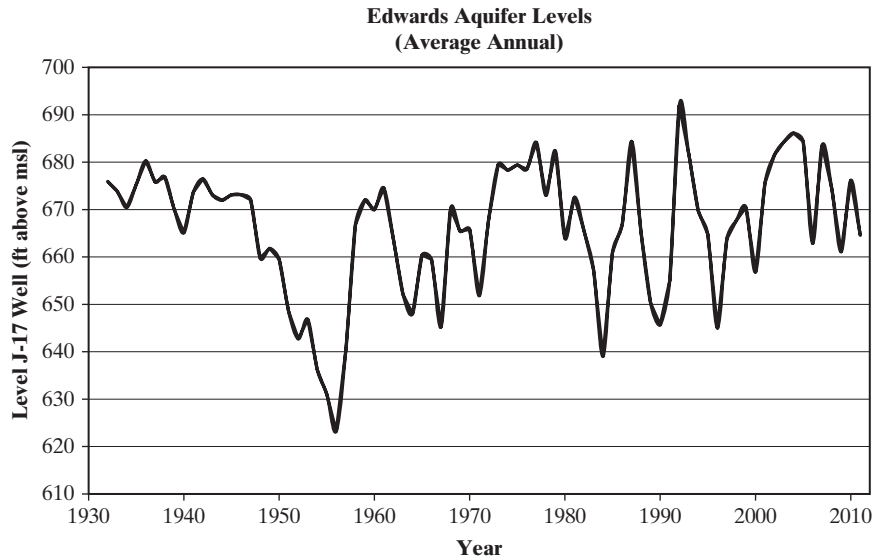
A major drought encompassed much of the United States from 1950–1956, beginning in the Southwest and covering 10 states by 1954 (NCDC, 2003). In Texas, drought conditions were so severe that the aquifer fell to the lowest levels ever recorded. Some farmers reported that rain had not fallen on their property for more than seven years and the San Marcos Springs, the largest artesian springs in Texas, ran completely dry, leading to intense concern over the future availability and management of the Edwards Aquifer as a water resource.

Water managers rely on monitoring wells to measure pressure within the aquifer. One of the well records, the J-17 Index Well in San Antonio, has been used since 1956 as an indicator of the health of the aquifer. The J-17 well is located on a major flowpath of the aquifer and thus responds quickly to pressure changes from pumping or recharge. Similarly, the levels in the J-17 well can be related to major artesian spring flows such as the Comal Springs and San Marcos Springs (Fig. 13–4). The absence of spring flow causes rivers to run dry and lake levels to decrease dramatically, and aquifer depletion can also cause severe environmental damage to the underground ecosystem. The Edwards Aquifer is home to many freshwater species of salamander, catfish, shrimp, and other organisms, eight of which have been placed on the endangered species list.

Historically, the Aquifer was subject to “right of capture” laws, allowing anyone with wells drilled into the aquifer to use unlimited water supplies. This has led to significant depletion of the resource, especially as the volumes drawn from the aquifer continually increase. In 1993, Senate Bill 1477 established the EAA to manage, conserve, preserve, and protect the aquifer,

Figure 13-4

Variations in Edwards Aquifer levels measured at the J-17 monitoring well in Bexar County.



and in 2001 the EAA began limiting pumping by issuing permits. Intending to protect spring flows by 2012, the Authority established junior and senior pumping rights. Thirty percent of the reductions were expected to be seen as permanent in 2001, and another 60% of reductions as permanent by 2008, depending on population increase. The permits are transferrable, allowing farmers and municipal water purveyors to buy, sell, and invest in withdrawal permits (Donahue and Klaver, 2005).

Aquifer management is one of the most pressing political discussions in central Texas today. Historically, water resource planning has been based on the drought of the 1950s, the worst drought on record. However, there is some discussion that with climate change more severe droughts could occur and managers would be unprepared to address the extreme water shortages that would transpire.

In 1998, the EAA mandated water-use cutbacks to address reduced spring flows and alarmingly low aquifer levels for the first time. By 2007, the EAA had developed a multiphase plan to reduce aquifer withdrawal during drought periods that met state mandates. The plan has four critical period stages determined by aquifer levels at the J-17 Index Well and the Comal and San Marcos Springs flows. Each critical period stage is triggered when the 10-day average aquifer level or spring flow is below the minimum value of the next stage. The critical stages from I to IV range from water levels < 660 ft msl, < 650 ft, < 640 ft, and < 630 ft msl and provide withdrawal reductions of 20%, 30%, 35%, and 40%.

Although the EAA mandates the percent withdrawal reduction, it does not determine how the cutbacks are met by the individual users. Cities relying on the Edwards Aquifer have created their own restrictions to meet

each critical stage reduction. For example, to reduce water consumption in San Antonio, Stage II restrictions prohibit the use of fountains, waterfalls, or other aesthetic water features both outdoors and indoors. More information about San Antonio's critical stage-based water restrictions can be found at www.saws.org/conservation/.

During 2011, La Nina conditions encompassed central Texas bringing hot, dry air to central Texas and virtually eliminating the hope for any rain early in the year. In July 2011, the EAA had declared Stage II drought restrictions (SAWS, 2011). The severity of this drought concerns water managers because, while it is not the longest drought on record, nor is the aquifer at its lowest historical level, it is the driest and hottest eight-month period on record since 1895 (LCRA, 2011). The Texas State climatologist declared this drought as the most severe one-year drought in Texas history. Usually severe droughts end in Texas after a severe storm or hurricane drops significant rainfall over the region causing aquifer levels to rise and often resulting in significant overland flooding.

Flash Flood Alley

Texas is accustomed to month- or year-long droughts, leading to intense water shortages throughout the state, as discussed in the previous sections. However, in some years, intense rainfall exceeding 10 inches per day leads to significant flooding. In fact, central Texas often receives most of its annual rainfall in a few large events rather than steady precipitation throughout the year. Thin soils, exposed bedrock and sparse vegetation, as well as steep slopes, produce some of the highest runoff rates in the country, making the Texas Hill Country the most flash flood-prone area in the United States.

The IH-35 corridor, spanning from Dallas to San Antonio, has been appropriately named Flash Flood Alley. Streams and rivers receiving runoff from the Texas Hill Country compounded with heavy rainfall cause massive flooding in the flat, gulf coast plains. Between 1959 and 2005, 760 flood-related deaths occurred in Texas, ranking it the number one for flood-related fatalities, injuries, and casualties (Ashley and Ashley, 2008).

In central Texas, peak discharges occur much more quickly than in other watersheds due to the slope and intensity of rainfall, catching residents off guard and putting them in the way of the flood. Floodplain maps are often subject to political pressures and may be inaccurate by the time they are finally made public. In Texas, floodplains are heavily disputed territory and become residential because of waterfront views. Many casualties occur when drivers attempt to traverse low water crossings while evacuating (Fig. 13–5). Since flooding is infrequent and often unexpected, central Texans assume that in their lifetime no significant flood will occur. However, in 1998 and again in 2002, flash flood alley lived up to its name.

Historically, unprecedented flooding occurred in south-central Texas in October 1998. Near perfect conditions caused torrential rains between

Figure 13-5

Flooding at a low water crossing behind Olmos Dam during October 2002 forced city managers to close US-281.



Austin and San Antonio, dumping up to 22 inches of rain at some locations (U.S. Department of Commerce, 1999). Flash flooding between Austin and San Antonio began on October 17, and by the evening of the 18th, seven river basins, draining approximately 10,000 square miles, were affected by flooding. Record peaks occurred at 15 river locations, and ground-water levels in monitoring wells remained 100 feet above the normal for two weeks after the storm.

The most notable flooding occurred in Cuero, Texas, about 80 miles southwest of San Antonio. Peak flows were three times those previously recorded, and the floodplain was 3 miles wide (U.S. Department of Commerce, 1999). Approximately 43% of homes were destroyed by the flooding, and property damage amounted to \$750 million. In addition, more than 10,000 people were displaced. It is important to note that this flood event was not out of the ordinary. On average, major flooding occurs on the Guadalupe River every five years.

In 2002, a stationary area of low pressure developed over central Texas. In early July, parts of the Hill Country received in excess of 30 inches of rain (Purpura et al., 2009). Significant flooding occurred on the Guadalupe, San Antonio, Atascosa, Frio, and Nueces Rivers. However, unlike the 1998 storm, the 2002 event dropped most of its rain above Canyon Lake. Canyon Dam was constructed in 1958 at the edge of the Balcones Escarpment to conserve water and to protect the lower Guadalupe River Basin from flooding. The reservoir can hold up to 728,400 ac-ft during flood conditions, but during 2002 water crested the dam spillway, and emergency managers feared the possibility of dam failure.

Canyon Dam provided some protection to downstream communities; however, the intensity of the rainfall and amount of water flowing from the Upper Guadalupe River was more than doubled the capacity of the reservoir. South of the dam, 800 homes were washed downstream (Eckhardt, 2011), many of which had been rebuilt after the 1998 flood event. This type of event could occur in a number of Texas cities, as so many are built along major rivers and streams. San Antonio was founded on the San Antonio River, and much of the city's downtown is prone to flooding. Consequently, significant engineering projects have been undertaken to protect downtown over the past decades.

San Antonio River

The San Antonio River has always been a unique resource for the city of San Antonio. Initially, it provided the water needed to sustain the San Antonio missions and the region's agricultural vitality. Today, the San Antonio Riverwalk is a tourist attraction and an economic resource. Over the years, citizens have debated the existence of the river and tried to develop the best management practices to address the many issues surrounding the river, such as drought, flood control, and water quality (Fig. 13–6).

The vitality of the San Antonio River is tied closely to the health of the aquifer and the amount of rainfall in a given year. The headwaters of the river originate at the Blue Hole, a famous spring north of downtown San Antonio. From there, the river passes through downtown before traversing gulf coast prairie for 240 miles, converging with the Guadalupe River and emptying into the Gulf of Mexico (SARA, 2011). It drains more than 4194 square miles.



Figure 13–6
Restaurants and
tour boats on the
famous Riverwalk
in San Antonio.

In the 1890s, when the citizens of San Antonio began relying on artesian and shallow wells for their water supply rather than surface water, the river level was significantly depleted. In response, the mayor of the City of San Antonio created the City Planning Committee in 1910 and began re-beautification efforts along the river banks. There were three primary proposals for beautifying the river, and the combination has created the basis for the Riverwalk today:

1. Thirteen miles of stone bridges and park along the river,
2. Channelize the river near downtown to control for flooding,
3. Build the shops along the river near downtown San Antonio designed to create a Venetian-like atmosphere to stimulate economic growth in the heart of San Antonio.

Re-beautification efforts met serious opposition when, between 1914 and 1921, six major floods occurred in downtown San Antonio. The worst of these occurred in 1921, when parts of downtown were inundated by as much as 10 feet of water. Significant loss of life and property prompted city officials to propose cementing over the river and turning it into a piped sewer system to prevent downtown from future floods. Luckily, the citizens of San Antonio protested and took other measures to protect downtown from flooding.

Olmos Dam was completed in 1926 at the cost of \$1.5 million and was built to contain water accumulating in the Olmos Creek watershed north of downtown. The area behind the dam has been turned into park and recreational facilities that flood during heavy rainfall. With a spillway length of 1152 feet, the dam was designed to hold vast quantities of water. There are six floodgates to release water into Olmos Creek at the downstream side of the dam, which provide significant protection for areas of northern San Antonio.

In 1929, the city completed a second flood control measure: the cutoff channel. The cutoff channel was designed to allow floodwaters to bypass the “Great Bend” portion of downtown and is controlled by a series of floodgates located at the entrance and exit to the great bend. When the river reaches a certain stage, the gates are closed and the water is diverted away from downtown. This portion of the river is home to shops and restaurants that act as the city’s main downtown attraction. Much like in Venice, residents and tourists can dine and shop along the waters edge.

The most impressive engineering accomplishment in the City of San Antonio was a tunnel 140 feet beneath the city that diverts floodwaters before they reach downtown and the Riverwalk, controlling the river stage in the downtown area. The project was completed in 1997 at the cost of \$111 million. The tunnel runs for 3 miles beneath the city and carries water downstream where wider channels can handle the excess floodwaters. The tunnel functioned perfectly during both the 1998 and 2002 storms when 30 inches of rainfall fell on the city in 48 hours. Had the tunnel not been in place at the time of the storm, the damage to the city could have been catastrophic.

In 1996, the San Antonio River Improvements Project was initiated. Approximately 13 miles of river, from Brackenridge park to the last mission in South San Antonio, is being restored at the cost of \$358 million. Engineers and urban planners re-evaluated floodplain extents and designed a green space consisting of pocket parks, paddling and bike trails, and wildlife habitat to provide flood storage and a recreational outlet for citizens. The unique culture of the city of San Antonio and its amazing Riverwalk is one of the premier urban water resources redevelopment projects in the United States and has drawn tourists from around the world for pleasure and conferences.

The Colorado River was once one of the wildest rivers in America, and it serves as the lifeblood of the American Southwest. For most parts of the year, the river gently meanders below the tall canyon walls, but between April and July three-fourths of the annual flow torrents downstream as snow in the mountains begins to melt. The river is often regarded as wild and uncontrollable because of violent and sometimes spontaneous floods caused by snowmelt. Powerful flows scour enormous amounts of reddish-brown sediment from the mountains and high plateaus, giving the river its original name, Río Colorado, or “red river” (USBR, 2006).

The river begins in the mountains of Colorado, flowing through the high plateaus of northern Arizona, down through the Black Mountains near Boulder City, and through northern Mexico into the Gulf of California (Fig. 13–7). Generally, the river flows south-southwest, curving through 1400 miles of high mountains, deep canyons, and scorching deserts. The river is fed by the Green, San Juan, Little Colorado, and Gila Rivers. The drainage basin includes 242,000 square miles of U.S. land and 2000 square miles in Mexico, encompassing 1/13 of the United States’ area, including seven states: Arizona, California, Colorado, Nevada, New Mexico, Utah, and Wyoming (Construction of Hoover Dam, 1976). Much of the land in the Colorado River basin was once considered desolate and useless, with the lower reaches passing through the driest and hottest regions of the United States. Today, the taming of the west through dams and irrigation channels has allowed enormous cities to flourish. However, environmental impacts have also resulted from these major projects.

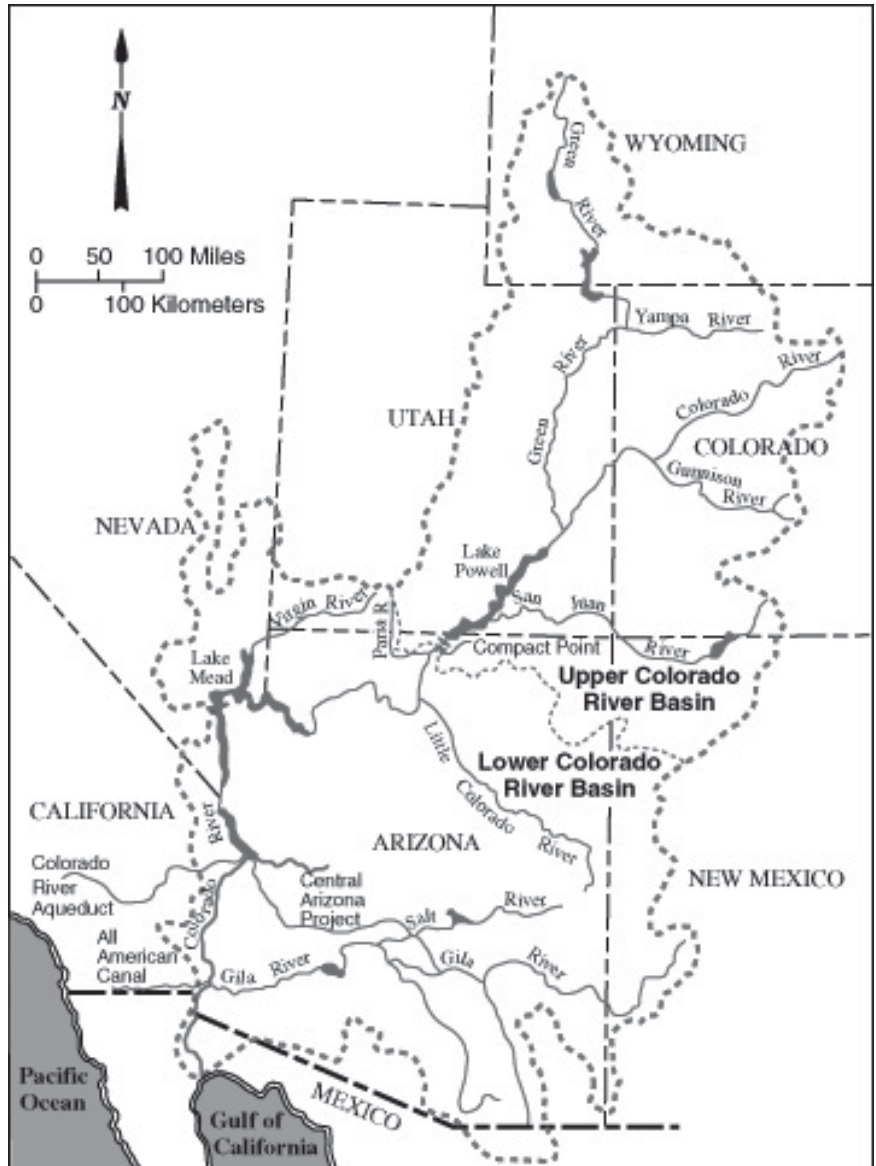
13.3 THE COLORADO RIVER—TAMING THE WILD WEST

Regional History

The first inhabitants to settle the Colorado River were Native American Indians, including the Hohokam people. The Hohokam people were a very sophisticated tribe living in what is now the state of Arizona. They were able to successfully divert water for irrigation from the Salt and Gila Rivers through 200 to 250 miles of canals, and they utilized the fertile red soil carried by the river and the constant source of water to irrigate their crops. In the late 1700s, Spanish explorers discovered the Colorado River and periodically began expeditions from the river’s mouth in the Gulf of California,

Figure 13-7

The Colorado River Basin in the southwestern United States.



but exploration upstream on the wild river was difficult and many explorers died in the extreme temperatures and rocky landscapes (USBR, 2006). In fact, the river basin was the last area in the contiguous United States to be explored, having unmapped and unnamed features as late as 1872 (Fradkin, 1996). When the American gold rush began in 1848, many travelers heading west traversed the basin and crossed the Colorado River where it meets the Gila River in southwest Arizona (USBR, 2006).

In 1896, the privately owned California Development Company began to construct irrigation canals to the Imperial Valley in Southern California. A canal was built along the old Alamo River, an overflow channel of the main river, and was routed up to the Salton Sea in California. The Salton Sea exists as a result of periodic flooding and retreating of Colorado River waters. After construction began on the Imperial Canal, Colorado River water finally reached the Valley in 1901 (The Bureau of Reclamation, 2006).

President Theodore Roosevelt began federal reclamation of the American Southwest by enacting the Reclamation Act of 1902 (USBR, 2006). The theory was to “reclaim” the arid lands for human use through irrigation (www.usbr.gov). The Act created the means for federally funded irrigation projects by setting aside money from the sale of public land, which would be used to build and maintain irrigation works in the western 13 states. The newly irrigated lands encouraged people to move west where they could grow crops on fertile soils, relying on a steady source of water. The irrigated land, now more valuable, was sold and the money put back into the revolving irrigation fund (Loring, 2004).

Reclamation was not an easy fix to conquering the west, as the unruly river and erratic flows provided many irrigation challenges in the early 1900s, especially in the lower stretches of the river. Flow in the Colorado is highly variable, fluctuating between 6 and 20 million ac-ft annually (Carriquiry and Sanchez, 1999). Peak flows coincide with the spring snow melt, but during other times of the year its flow can be much lower, limiting its ability to irrigate all the dependant farmland. Also, the violent spring floods require sufficient and expensive levees to contain and direct the water, while at the same time the highly turbid water deposits enormous amounts of sediment. It was estimated that the river in its natural, red-colored state carried 160 million tons of sediment each year (Carriquiry and Sanchez, 1999). This massive load of sediment clogged man-made canals after a large flood, and required expensive repairs each time.

Aside from flood control problems, water salinity is the biggest water quality issue in the Colorado, and can result in completely useless water. Salinity concentrations in the headwaters of the Colorado are less than 50 mg/L, but concentrations increase downstream dramatically, reaching about 900 mg/L near the Mexico border (pubs.usgs.gov). Many of the minerals come from the saline soils in the high plains of Colorado and Arizona, but agricultural irrigation also is a significant contributor. In fact, water flowing in the lowest reaches of the river has been re-used, on average, three times for irrigation purposes, allowing it to leach more minerals from the soil and become more saline (Fradkin, 1996). Urbanization, population growth, mining, agricultural practices, and recreation all affect salinity concentrations and other chemical constituent concentrations in the Colorado River (pubs.usgs.gov).

For the people of the Southwestern United States, the Colorado River in the late 1800s needed to be tamed. Its wild, seasonal flooding and receding

impacted farming, and this unpredictability prompted the people of the Southwest to search for a solution. In the spring of 1905, a series of floods caused the Colorado River to overflow into the Imperial Canal, and eventually changed the course of the entire river into the Salton Sea. Several weak dams failed, allowing the floodwaters into the Imperial Canal, eroding the soft channel deeper and deeper until it carried the entire river. For 18 months the Colorado River changed its course, and the Southern Pacific Railroad Company paid 3 million dollars to direct the river back into its main channel (USBR, 2006).

The Southwest saw how vulnerable they were to the power of the river, and pleaded to the federal government for help. The Department of the Interior's Bureau of Reclamation (USBR) was given the responsibility of this task. After examining the problem, it proposed that constructing a dam to control the flow of the river would not only alleviate the seasonal flooding and drought problems the people faced, but it would also provide the vital freshwater that the area needed to grow. Before the construction on the dam could commence, the USBR needed to solve a major legislative issue that was blocking its path: water rights on the Colorado River.

Withdrawal Regulation

A commission was created in August 1921 and was headed by the Secretary of Commerce Herbert Hoover to address the issue of water rights. It comprised representatives from the seven basin states affected by the construction of the Hoover Dam: Colorado, Wyoming, Utah, New Mexico, California, Nevada, and Arizona. The seven basin states debated extensively over water allocations, but could not settle on an agreement. By Hoover's suggestion, an agreement was finally reached that apportioned the water between two groups, the Upper and the Lower Basin States. This put forth the movement to establish the first water rights on the river, and water rights for the individual states would be allocated in future agreements. The Colorado River Compact, also referred to as the Hoover Compromise, was signed in 1922 officially dividing the basin into two parts near the Arizona/Utah border (USBR, 2006).

The compact allotted to each basin 7.5 million ac-ft of water to be used for beneficial consumptive purposes each year. This volume of water was then to be divided among the states in each basin. On October 11, 1948, the states in the Upper Basin signed the Upper Colorado River Basin Compact. Colorado was given the largest sum of water with 51.75%, Utah with 23%, Wyoming with 14%, and New Mexico with 11.25% (USBR, 2006). The Lower Basin States, however, were unable to willfully reach a compromise, and a suite was filed in the U.S. Supreme Court in 1952 by Arizona. Arizona asked for the Supreme Court to determine how the river's water should be divided among the three lower states. Finally, in 1964 a decree was established by the Court stating that California was

entitled to 4.4 million ac-ft, Arizona to 2.8 million ac-ft, and Nevada to 0.3 million ac-ft (The Bureau of Reclamation, 2006). Mexico was entitled to 1.5 million ac-ft annually through a separate treaty. This made the Colorado River the most highly litigated and politicized river in the country (Fradkin, 1996).

Hoover Dam

Having established the water rights among the basin states, the USBR could now choose a location to build the dam. After exploring 70 potential sites with the help of the U.S. Geological Survey, the Bureau identified two canyons, Boulder Canyon and Black Canyon, which were feasible locations for the dam. In 1928, after further data collection, Black Canyon was chosen for three reasons: the shallow depth to the bedrock eased excavation, the dam at Black Canyon would store the same amount of water at a lesser height, and the geological structure of the area was deemed more favorable for construction (Fig. 13–8). With the site chosen, President Calvin Coolidge signed and enacted the Boulder Canyon Project Act on December 21, 1928.

The Hoover Dam and the resultant reservoir, Lake Mead, would serve four main purposes once completed: flood control, water storage to allow for irrigation and domestic use, downstream silt control, and electricity generation. The construction of the Hoover Dam was awarded to Six Companies,

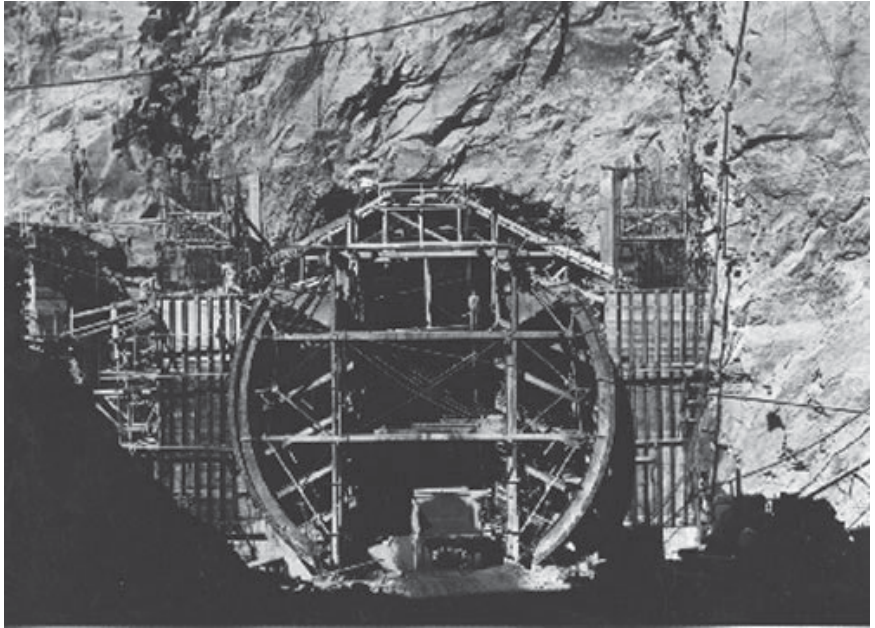


Figure 13–8

Black Canyon before the construction of Hoover Dam on the Colorado River.

Figure 13–9

A diversion tunnel built for Hoover Dam.



Inc., an amalgam of six major contracting firms located out west, which proposed the job could be done for \$48.9 million. To put the enormity of the bid in perspective, the amount was roughly a quarter of the nation's annual public works project budget at that time and created the largest construction project ever undertaken.

In June 1931, workers began diverting the Colorado River. Four diversion tunnels 56 feet in diameter and nearly 3 miles in total length were blasted through the walls of the canyon (USBR, 2006) (Fig. 13–9). When the tunnels were completed in November 1933, engineers redirected the flow of the river using two temporary cofferdams made of rock and earth to lift the water into diversion channels. The upper cofferdam was built just below the opening of the four tunnels to divert the flow of the Colorado River down the tunnels. On November 14, 1932, the flow of the Colorado River successfully wound its way around the work site, and the task of excavating the construction site began.

Engineers were tasked with designing spillways, intake towers, penstock and outlet systems, and powerhouse and switching stations as well to hydroelectric generation. These components were no less important and very often no less difficult to construct than the dam face. The spillways located on each side of the dam were responsible for handling excess water flow in the event that flooding conditions were present in Lake Mead, the reservoir created behind Hoover Dam. At capacity, the concrete lined channels running 650 feet long and 125 feet wide could handle 400,000 cubic feet per second of overflow rushing at 120 miles per hour (Construction of Hoover Dam, 1976).



Figure 13–10
An aerial view
of the completed
Hoover Dam and
Lake Mead.

It was also necessary to design a way to move the water from Lake Mead through the Hoover Dam. Engineers designed a series of intake towers and penstocks that would pull water from the reservoir through 41 feet tunnels lined with 2 feet of concrete and massive thick plate steel pipes. The intake towers sat on ledges that were cut out of the canyon approximately 260 feet above the river bed in order to provide a basin for silt settling. The towers would be connected to tunnels that would drop 200 feet straight down and connect to a series of underground tunnels that flowed into the dam.

On February 1, 1935, engineers began filling the Hoover Dam, and within two months the crest was reached. In less than five years, two years ahead of schedule, the Hoover Dam stood completed. The completion of the Hoover Dam was an engineering success the likes of which the world had never seen and it served as a symbol of hope for the people of the Great Depression. The Hoover Dam towers 726.4 feet above the bedrock, spans 1224 feet between the canyon walls, holds back 22.5 tons of pressure at the base of the dam, and weighs a staggering 6,600,000 tons (Fig. 13–10). Lake Mead became the largest man-made reservoir in the U.S., holding a massive 25.9 million ac-ft of water, stretching 110 miles upstream and covering 157,900 acres (USBR, 2006).

The Hoover Dam has become an integral part of the Southwestern United States. The water stored behind its walls in Lake Mead is used daily to service 20 million people through domestic, industrial, and municipal purposes.

The water that surges through its 17 commercial generating units, with a 2078 MW capacity, supplies enough clean electricity to serve 1.3 million people (USBR, 2006). As a flood control device, the Hoover Dam has been successful. While it cannot prevent every major flood event, it has helped to regulate the flow by alleviating flooding between February and June and drought between June and February.

However, Hoover dam has also contributed to environmental degradation. For example, flow regulation has disrupted wildlife habitats and fish migration patterns in the region. The dam has also trapped much of the sediment that used to flow downstream. This has increased erosion downstream. Within nine years of operating, the thickness of the downstream riverbed decreased by more than four meters, lowering groundwater tables and threatening local wells and vegetation.

Glen Canyon Dam

After the construction of the Hoover Dam, the government needed to provide a means with which water could be regulated to ensure that the Lower Basin and Upper Basin states would receive their rightful allotments of water. After negotiating with the Sierra Club regarding multiple dam sites, Congress authorized the Colorado River Storage Act on April 1956, which ensured the delivery of allotted water downstream. The construction of the Glen Canyon Dam in northern Arizona began on June 1960, and the final concrete was poured on September 1963. The Glen Canyon Dam towered over the landscape at 710 feet tall, and created a reservoir, Lake Powell, with 26 million ac-ft of storage stretching over 186 miles of the Colorado River that took 17 years to fill. The Glen Canyon Dam is infamous for the serious environmental damage it has caused (www.usbr.org).

David Brower, the executive director of the Sierra Club at the time of completion, called the Glen Canyon Dam “America’s most regretted environmental mistake.” In fact, Glen Canyon was one of the most beautiful canyons along the Colorado River before it was inundated with water (Fig. 13–11). Post-construction, numerous environmental concerns have arisen. For instance, under natural conditions the flow through the Grand Canyon would range between 3000 and 90,000 cubic feet per second and carry massive amounts of silt. Now that the Glen Canyon Dam is regulating water flow, the annual flows range between 8000 and 20,000 cubic feet per second and are virtually sediment free (livingrivers.net). The natural high and fluctuating flows served as a conveyor of sediment to build beaches, rejuvenate and replenish the shorelines, as well as create sandbars that supported calm, backwater areas for fish to utilize, while the low flows were essential in juvenile fish survival. Instead, the fluctuating, often lower, sediment-free flows seen erode the sand bars and allow vegetation to encroach upon areas that are not naturally vegetated. The disruption of this natural system has significantly impacted Grand Canyon National



Figure 13–11
An aerial view
of Glen Canyon
inundated by Lake
Powell behind
Glen Canyon
Dam.

Park where some species have become extinct and canyon flows have been reduced to a trickle.

Sediment and flow issues are not the only environmental problems created by the Glen Canyon Dam. Because the reservoir is so deep, the water released from the lower portions of the reservoir exits the dam at fairly constant 47°F. The natural water temperature of the river flowing through the Grand Canyon should have a wide range, from nearly freezing in the winter to nearly 80°F in the summer. The natural temperature fluctuation is important in various biological processes which now must deal with extra stress.

Glen Canyon Dam is wasteful of significant volumes of water each year. Lake Powell alone loses 2600 gallons of water per minute to infiltration in the canyon walls (www.usbr.gov), nearly 0.86 million ac-ft of water annually. While this is a small number when compared to Lake Powell's total capacity (around 3%), it constitutes nearly 6% of the river's annual flow, and together with Lake Mead the two reservoirs account for a 10% loss of flow (glencanyon.org). In a region that relies heavily upon the Colorado River for multiple purposes, water lost to evaporation is simply unsustainable.

In 1983, a greater than average snowmelt caused significant flooding upstream of Glen Canyon Dam. As upstream dams began releasing their flows, the Glen Canyon Dam became overwhelmed. Engineers opened the spillways on either side of the dam, but rumbling and shaking caused immense concern. Concrete, rebar, and sandstone flew out of the spillway tunnels at the base of the dam, but to avoid failure, the spillways had to

be kept open (Hannon, 2003). If Glen Canyon Dam were to have failed, a 580 ft wave would have overtopped Hoover Dam (Living Rivers, 2005).

Some discussion has arisen about the removal of Glen Canyon Dam. Proponents argue that in the wake of the 1983 flood, dam failure is inevitable. Additionally, Hoover Dam has the capacity to provide the equivalent storage of Glen Canyon Dam. Removing the dam would alleviate multiple environment concerns and return a significant portion of the river back to its natural state. For the case of Glen Canyon Dam, the negative impacts may far outweigh the benefits provided by the dam.

The Colorado River runs deep into the United States, both geologically and historically. Initially the wild river impeded exploration and development of the Southwest, and without a reliable and controlled source of freshwater, the U.S. Southwest would likely remain a desolate and vast desert. Great engineering feats, such as the Hoover Dam, have allowed humans to tame the Colorado River and provide sources of freshwater to its inhabitants and to agricultural interests. However, the river must be carefully managed so that it is not overused to the point that floods, droughts, salinity, and other environmental impacts drive people away.

13.4
ACROSS THE
POND—THE
RIVER THAMES

The Thames River, one of the most famous in all of Europe, flows 205 miles from the Thames Head, in the Cotswold Hills, through Southern England before emptying into the North Sea near London. It is the longest river in England and drains approximately 5500 square miles. The last 65 miles of the river are an estuary where the salt water fresh water barrier fluctuates daily. In London, the incoming tide can raise the river up to 7 meters during the spring, reversing its flow through the estuary (Encyclopedia Britannica, 2011).

Historically, this river suffers from overuse and neglect. Runoff from waste and sewage has severely contaminated the river for decades, a problem that has only recently been addressed by authorities. In addition, severe storms in the North Sea can inundate London with storm surge. Storm surge is the phenomena brought on by high winds and shallow bathymetry when a wall of water is pushed ahead of a storm and onto land. To combat these issues, engineers have constructed a series of water resource projects that mitigate the effects of contamination and flooding.

Much of the early Victorian sewer network is still in use today for the movement of water and contaminants, but as London continues to grow, the system becomes ever more taxed. During heavy rains, the sewer system overflows, affecting plant and animal life and presenting a serious human health risk. The Thames Tideway Scheme is a project currently in progress that introduces two new storage-and-transfer tunnels as well as capacitance improvements to three wastewater treatment plants. The tunnels will reduce overflow into the River Thames and Lee, moving it to the treatment facilities instead. The Thames Tunnel is to be constructed between 2013 and 2020. It

will be more than 20 miles long and 25 feet in diameter, flowing parallel to the River Thames. The tunnel is designed to remove 96% of the sewage entering the Thames annually (NRI, 2011).

Flooding and the Barrier

Floods on the River Thames often occur because of large storms, upstream flooding, extreme tidal movement, or severe storm surge. The combined effect of any of these can be catastrophic. In 1928, 14 people died when all four causes occurred simultaneously (Encyclopedia Britannica, 2011). Snow-melt and unusually heavy rainfall coupled with spring tides and storm surge filled London's streets with four feet of water. Thousands of people lost their homes and the city's infrastructure was significantly damaged.

In 1953, another major storm in the North Sea damaged the English coastline and sea walls. Floods carried entire structures out to sea, leaving 24,000 properties seriously damaged. In the United Kingdom, 300 people died because of the storm. These significant events, along with smaller ones, prompted the construction of the Thames Barrier, the world's second largest movable flood barrier (completed in 1982). Built downstream of London, it spans 1700 feet across, and protects over 33,000 acres from flooding (Environment Agency, 2011a).

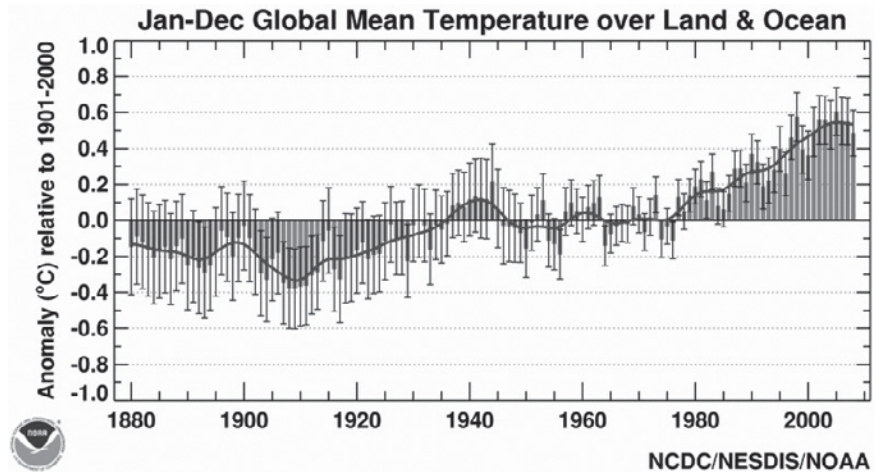
The Thames Barrier is made up of 10 steel gates, 65 feet high. There are two types of gates: falling radial gates and rising sector gates. The falling radial gates are situated above the non-navigable portions of the river while the rising sector gates rest on the river bed, allowing ship traffic to pass. The gates can be maneuvered into four positions: open, closed, under spill, and maintenance. The entire structure can be readied for flooding within 90 minutes (Environment Agency, 2011a).

The flood barrier protects significant infrastructure, including historical landmarks, government buildings, businesses, schools, hospitals, etc. However, it leaves the entire downstream Thames estuary vulnerable to severe storms. Estuaries have significant capacity to store storm surge and help mitigate the effects of severe storms. In 2002, England established the Thames Estuary 2100 Plan, a flood risk management plan that takes into consideration climate change and socioeconomic change. The Plan would allow the river to reclaim its floodplain near the estuary to reduce the impacts of flooding in London (Environment Agency, 2011b). This non-structural alternative reflects opinions of water managers worldwide and is one effort to consider the true future impacts of climate change in Europe.

Over the past one hundred years, global average surface temperatures have increased. The rate has accelerated since 1970 and 11 of the 12 years between 1995 and 2006 were the warmest on record (Solomon et al., 2007). Despite ever-continuing arguments about whether climate change exists and whether this is a natural climatic shift, it is generally well-accepted that temperatures

Figure 13–12

Variations in global mean temperatures over land and ocean between 1880 and 2010.



and weather patterns around the world are changing (Fig. 13–12). Glaciers are melting at extreme rates, storms are more intense, and dry spells in some parts of the world are unprecedented according to the IPCC (Solomon et al., 2007).

The Intergovernmental Panel on Climate Change (IPCC) was established by the United Nations Environmental Programme and the World Meteorological Organization to provide the world with a scientific view of climate change and its potential socio-economic and environmental impacts. The IPCC reviews the work of thousands of scientists from the international pool of the best engineers, atmospheric scientists, chemists, physicists, etc. who voluntarily conduct research on climate change. The combined efforts of their research show that temperature increase and other weather patterns seem to correlate with increased atmospheric carbon, mostly from human sources.

Conclusions drawn by the IPCC indicate that increases in land and sea surface temperature are already beginning to alter the hydrologic cycle. Higher temperatures lend to increased evaporation and higher water vapor content over oceans. Subsequently, weather systems are becoming larger and precipitation events more severe, even though annual precipitation is reduced. Such change presents a challenge to hydrologists when determining how to develop sustainable communities through water resource projects.

Interactions of Human and Natural Water Systems

Across the world, drought, exacerbated by increased populations, is taxing available freshwater resources. Rivers and aquifers that were once plentiful are no longer being fully recharged in the face of decreased annual precipitation. Countries, states, cities, and communities will argue or even go to war over the remaining resource. In response, authorities are continuously looking for ways to create long-term water storage. However, structural solutions, such as dams and their reservoirs, may do more harm than good in certain regions.

Like the Glen Canyon Dam, many dams across the United States have severely impacted river systems and are being heavily protested. On the Elwha River, near Olympia in the state of Washington, the largest dam removal project in American history has begun in 2011 (Yardley, 2011). A 210-ft tall hydroelectric dam, built in 1912, has so greatly impacted the salmon population in the river that environmental groups have successfully litigated to condemn the dam. Its removal will hopefully bring back the salmon population in the river. Similarly, many of dams on the Columbia River system in the Pacific Northwest have potentially decimated salmon populations, but the 14 hydroelectric dams on the river produce more hydroelectric power than on any other North American river (American Rivers, 2011). Water can yield incredible power, but often the negative environmental impacts outweigh the economic benefits. Part of future water resources planning will be weighing the true costs and benefits of proposed projects. Thus, instead of the old traditional economic benefit/cost analysis used to analyze major dam and reservoir projects in the past, true environmental costs need to be more accurately measured.

Internationally, dams are becoming ever larger and more impactful. Climate change and the eventual depletion of petroleum resources are contributing to a massive hunt for alternative sources of power for countries in Asia. China as a vast country has relatively abundant water resources and is ranked sixth in the world after Brazil, the Russian Federation, Canada, the United States, and Indonesia in terms of absolute amount of annual runoff. Figure 13–13 shows three major Chinese rivers: the Yellow River, the Yangtze River, and the Mekong River. Due to the dominance of the monsoon climate, the temporal and spatial distribution of water resources is highly variable. Flooding as one of the major water problems in China has brought great attention from the government and public over the years.

The Yangtze River flows for 3988 miles from the glacier on the Tibetan Plateau eastward across southwest, central, and eastern China before emptying into the East China Sea at Shanghai. It is the third longest river in the world behind the Nile and the Amazon. The Yangtze River's notorious floods in southern China have been recorded for millennia and have claimed more than 1 million lives in the past century. In order to control flooding and develop a future source of energy, China began constructing the Three Gorges Dam in 1994. Completed in 2006, at a cost of \$28 billion, the dam is 1 mile wide and more than 600 feet high and creates a reservoir hundreds of feet deep and nearly 400 miles long. Chinese authorities claim that the dam's hydropower turbines are able to create as much electricity as 18 nuclear power plants. The project has been a source of pride for the government, but has aroused intense debate among scientists.

While the gigantic dam has benefitted the region, there have also been a variety of issues since construction began in 1992. More than 1.4 million people were forced to relocate and resettle. More than 1000 towns and villages were inundated by the reservoir, and landslides and pollution have



Figure 13–13

Major rivers of China: the Yellow, Yangtze, and Mekong Rivers.

plagued areas near the dam since it was built. Additionally, the dam can cause rapid silt build-up in the reservoir, affecting both upstream and downstream ecosystems of essential nutrients. In a rare admission, in 2011, the Chinese government stated that the Three Gorges Dam, the world's largest hydropower plant, is having “urgent problems” and warned of environmental, construction, and migration “disasters” amid the worst drought to hit southern China in 50 years. The Chinese government also admitted the Three Gorges Dam has negatively impacted downstream river water, transport, and migration (Kent, 2011).

In southern China, eight hydroelectric dams have been proposed on the Mekong River, which will be capable of storing half of the river's flow.

Despite the incredible electric capacity of these dams, they have been heavily protested. After leaving China, the Mekong River flows for 2800 miles through Southeast Asia and into Cambodia and Vietnam, providing food to over 60 million people (Pearce, 2009). The potential political and environmental impacts of damming the Mekong River are huge. Not only will China have complete control over the flow of the river to its downstream neighbors, but the dam will greatly reduce the productivity of the fishing industry, the lifeline of these south Asian countries. The Tonle Sap, a tributary to the Mekong, is the world's second largest inland fishery. In summer, the Mekong's annual flood reverses the flow in the Tonle Sap contributing to its productivity. The Chinese dams have the capacity to forever destroy this ecosystem and its productivity.

Often structures can exacerbate the impacts of natural disasters. For example, the Mekong River is located near a significant fault in central China. Engineers believe that the extreme weight of the water stored behind some of the dams can accelerate, if not cause, earthquakes along this fault. Whether this phenomenon, called induced seismicity, may have played a role in the 2008 earthquake in Sichuan China is highly disputed. Similarly, if a dam were to fail during an earthquake, the ensuing floods would be disastrous.

The interaction between human and natural systems is significant when planning water resource projects. Not only can an ill-advised or insufficiently planned project negatively influence a resource, but may also encourage unsustainable development. In any given location, natural disasters can occur infrequently, but when they do, the impact is felt for years afterwards. As populations and communities continue to grow and expand, the number of people living and moving into vulnerable areas increases.

Sustainable Development and Coastal Areas

In coastal areas of the United States, 87 million people, or 29% of the U.S. population, live in coastal counties, a 40 million person increase since 1960 (Wilson and Fischetti, 2010). In any given year, a hurricane could destroy much of the infrastructure along the coast. Scientists now have the ability to predict extreme events and often give communities the lead-time they need to evacuate or find shelter (Chapter 12). However, coastal community resiliency still baffles engineers especially in the face of climate change, sea level rise, and increased storm severity. The IPCC has openly published that since the 1970s the potential destructiveness of hurricanes has increased, trending toward longer and more intense storms. Water resource planners must find innovative new ways to protect coastal communities that are often a combination of structural and non-structural solutions.

As we continually encroach on the natural system, man-made environmental disasters become more prevalent. The residual effects of such disasters can impact water resource systems for years afterward. Large oil spills, such as Exxon Valdez, 1989, and the BP Oil Spill, 2010, had huge short-term

environmental and economic impacts, but the long-term impacts of these events are often overlooked. The numbers of fish, birds, and other wildlife, as well as the marshes and beaches, effected by oil spills are enormous and the costs of the permanent cleanup astronomical. Such major events receive more attention than the smaller spills of heavy metals, organic chemicals, and fertilizers, but these spills can have lasting impacts on water supply systems, both surface and groundwater. Economic activities to support population growth can potentially play a huge role in the future quality of our water resources and our ability to provide for future generations.

Dams that destroy entire ecosystems, major spills and leaks, and global warming that puts the world as we know it at risk, if nothing else, have shown that there is a limit to how much the natural system can be abused before the impacts are irreversible. Until recently that limit seemed unattainable, but as populations keep growing, the carrying capacity of the earth will eventually be reached. Future water resource projects must be developed in a way that is sustainable so as not to permanently destroy our access to water.

Sustainable development, “development that meets the needs of current generations without compromising the ability of future generations to meet their own needs” (United Nations, 2009), is a concept that has recently emerged as a necessary component of water resources planning. Sustainability is often referred to as a three-legged stool on which our continued and future existence sits. Each leg represents one area of sustainability: economic, social, and environmental. The interactions of these three areas and the balance act required to sit levelly on the stool determine how sustainable a project or a lifestyle is.

As discussed in previous sections, some projects are completed without due consideration of all associated impacts. The over-allocation of water resources to burgeoning populations in developing countries will only further damage the environment and lead to political upheaval. According to the critically acclaimed book *Collapse* (Diamond, 2005), peaking populations, coupled with climate change and a decrease in available water supplies, led to the Mayans demise hundreds of years ago. Similarly, unsustainable and uncontained growth may trigger the collapse of modern societies, as in many of the drought-ridden areas of Africa and the flood-prone and contaminated regions of India. In the future, it will be extremely important for engineers and water planners to conscientiously evaluate the impact of a project from an overall sustainability point of view.

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Appendix A

Symbols and Notation

Symbol	Definition	Symbol	Definition
A	cross-sectional area of channel	D_0	median raindrop diameter
A	drainage area	d_p	depression storage
A	loss rate	DRH	direct runoff hydrograph
ADI	alternating direction implicit method	DRO	direct runoff
API	antecedent precipitation index	E	evaporation
$A_r(H)$	surface area of detention basin as a function of head	E	specific energy
A_T	total area	e	vapor pressure
B	channel top width	$E(x)$	expectation, mean, first moment
B	time of fall (SCS unit hydrograph)	e_a	vapor pressure of air
b	aquifer thickness	e_s	saturation vapor pressure
b	bottom width of channel	ET	evapotranspiration
b'	thickness of aquitard	F	field capacity
BF	baseflow	F	infiltration volume
C	Chezy coefficient	f	infiltration capacity or rate
C	runoff coefficient	$F(x)$	cumulative density function
c	wave celerity	$f(x)$	probability density function
C	radar design constant	$F(z)$	normal CDF
C_0, C_1, C_2	Muskingum constants	f_c	ultimate infiltration capacity
CDF	cumulative density function	F_f	frictional force
CN	SCS runoff curve number	F_G	gravitational force
C_p	Snyder's storage coefficient	F_H	hydrostatic force
C_{s1}	skew coefficient	f_0	initial infiltration capacity
C_t	Snyder's timing coefficient	Fr	Froude number
CV	coefficient of variation	F_{RM}	mean field bias correction
C_w	weir coefficient	G	ground water flow
D	duration of unit hydrograph	g	skew
D	Muskingum constant	G_i	gage rainfall accumulations
D	rainfall duration	GW	subsurface seepage to ground water flow
d	distance to water divide	H	heat storage
D	diameter of raindrops	H	relative humidity
D	average difference between gage and radar rainfall accumulation	H	total head
dBZ	decibels of radar reflectance	h	head
D_c	circular channel diameter	h	flow depth
DHM	melt coefficient	H_3	change in water surface elevation through bridge
D_i	diameter of size of category i	h_l	head loss
D_{max}	maximum raindrop diameter	h_0	weir crest elevation

Symbol	Definition	Symbol	Definition
I	heat index	N_i	number of raindrops in size category i
I	inflow		per unit volume
I	percent imperviousness	N_m	monthly daylight adjustment factor
i	natural piezometric slope	N_0	number of drops at diameter zero
i	rainfall intensity	O	outflow
$i(t)$	instantaneous inflow	P	pressure
I_a	initial abstraction	P	precipitation
IDF	intensity-duration-frequency	P	wetted perimeter
i_e	rainfall excess	p	probability
IUH	instantaneous unit hydrograph	$P(X_i)$	probability of X_i
J_0	solar constant	PDF	probability density function
K	conveyance factor	P_e	effective runoff
K	hydraulic conductivity	PE	potential evaporation over a 10-day period
K	routing parameter	PET	potential evapotranspiration
k	intrinsic permeability	PMF	probability mass function
k	kinematic flow number	P_r	radar-measured power
K'	vertical hydraulic conductivity of aquitard	Q	outflow
$K(\theta)$	capillary conductivity	q	flow per unit thickness
K_1	diffusion equation constant	q	infiltration rate
K_s	hydraulic conductivity at saturation	$Q_{(i,j)}$	recharge or pumping at node (i, j)
KS	Kolmogorov-Smirnov statistic at confidence level alpha	Q_1	inflow
K_x	horizontal hydraulic conductivity	Q_2	outflow
K_z	vertical hydraulic conductivity	Q_c	channel flow
L	channel length	Q_d	direct solar radiation
L	distance to wetting front	Q_e	energy of evaporation
L	evaporation capacity	Q_h	sensible heat transfer
L	lag time	Q_i	inflow
L	loss rate	Q_N	net radiation
L	attenuation loss	Q_O	daily radiation at top of atmosphere
L_c	latent heat of condensation	q_0	overland flow
L_c	length along channel to point nearest centroid of area	QOBS	observed hydrograph ordinate at time i
L_{ca}	length to centroid of drainage area	Q_p	peak flow
L_e	latent heat of vaporization	Q_T	peak flow rate for T -yr flood
L_f	latent heat of freezing	q_t	discharge at time t
L_m	latent heat of melting	Q_v	advected energy of inflow and outflow
L_s	infiltration storage	R	Bowen ratio
M	daily snowmelt	R	hydraulic radius
M	rate of heat storage	R	recharge
m	mass	R	surface runoff
MIT	minimum interevent time	r	radius
N	effective roughness	r	range to radar target
n	Manning's roughness coefficient	R	rainfall rate
n	porosity	r_c	radius of core sample
		RD	relative dispersion

Symbol	Definition	Symbol	Definition
RF	rainfall	w	channel width
R_i	radar rainfall accumulations	$W(u)$	well function
RMS	root mean square	WS	water surface elevation
RO	runoff	X	event
R_s	average albedo	x	horizontal distance
S	channel slope	x	Muskingum weighting factor
S	energy gradient	X_i	<i>i</i> th possible outcome
S	potential abstraction	x_m	median
S	storage	y	average watershed slope
S	storativity	y	depth
S	slope	y_c	critical depth
s'	drawdown	y_n	normal depth
S_{av}	average capillary suction	y_0	average depth of overland flow
S_c	critical slope	z	elevation
S_D	soil moisture storage	z	horizontal dimension for side slope of a channel
S_f	friction slope	z	standard normal variate
SMELT	melt rate	Z	radar reflectivity
S_0	channel slope	zi	thickness of layer <i>i</i>
S_0	overland slope	λ	wavelength
S_s	specific storage	λ	exponential distribution parameter
S_x	standard deviation of x	Γ	gamma function
T	return period	Λ	related to median raindrop size and rainfall rate
T	temperature	α	kinematic wave coefficient
T	transmissivity	α	attenuation factor
T	transpiration	α	compressibility of aquifer material
t	time	α	conveyance factor
T_a	temperature of saturated air	α	Gumbel distribution parameter
T_B	duration of flood wave	β	compressibility of fluid
T_b	time base of hydrograph	γ	psychrometric constant
t_c	time of concentration	γ	Gumbel distribution constant (Euler's constant)
T_d	dewpoint temperature	γ	specific weight of water
t_p	time to peak	μ	dynamic viscosity
T_R	time of rise	μ	mean value
u	wave velocity	θ	volumetric moisture content
$u(t - \tau)$	weighting function	ρ	fluid density
UH	unit hydrograph	ρ_b	bulk density
V	average velocity	ρ_m	density of moist air
V	total volume	ρ_w	vapor density
v	velocity	σ	standard deviation
Var(x)	variance, second moment	σ^2	variance
V_D	rainfall velocity	τ	incremental time unit
V_s	seepage velocity	Ψ	stream function
W	recharge intensity		
W	weight		
W	width of overland flow		

Appendix B

Conversion Factors

Multiply the SI Unit			To Obtain the U.S. Customary Unit	
Name	Symbol	by	Symbol	Name
Energy				
Kilojoule	kJ	0.9478	Btu	British thermal unit
Joule	J	2.7778×10^{-7}	kW·h	kilowatt-hour
Joule	J	0.7376	ft·lb _f	foot-pound (force)
Joule	J	1.0000	W·s	watt-second
Joule	J	0.2388	cal	calorie
Flow				
Cubic meter/second	m ³ /s	35.315	cfs	cubic feet/second
Cubic meter/second	m ³ /s	22.824	MGD	million gallons/day
Force				
Newton	N	0.2248	lb	pound force
Mass				
Gram	g	0.0353	oz	ounce
Gram	g	0.0022	lb	pound
Kilogram	kg	2.2046	lb	pound
Power				
Kilowatt	kW	0.9478	Btu/s	British thermal units per second
Kilowatt	kW	1.3410	hp	horsepower
Watt	W	0.7376	ft·lb _f /s	foot-pounds (force) per second
Pressure (force/area)				
pascal (newtons per square meter)	Pa (N/m ²)	1.4504×10^{-4}	lb _f /in ²	pounds (force) per square inch
pascal	Pa (N/m ²)	2.0885×10^{-2}	lb _f /ft ²	pounds (force) per square foot
pascal	Pa (N/m ²)	2.9613×10^{-4}	in.Hg	inches of mercury (60°F)
pascal	Pa (N/m ²)	4.0187×10^{-3}	in.H ₂ O	inches of water (60°F)
pascal	Pa (N/m ²)	1×10^{-2}	mb	millibar
kilopascal (kilonewtons per square meter)	kPa (kN/m ²)	0.0099	atm	atmosphere (standard)
Velocity				
kilometers per second	km/s	2236.9	mi/hr	miles per hour

Multiply the SI Unit		by	To Obtain the U.S. Customary Unit	
Name	Symbol		Symbol	Name
meters per second	m/s	3.2808	ft/s	feet per second
foot per second	ft/s	0.3048	m/s	meters per second
mile per hour	mph	0.4470	m/s	meters per second
mile per hour	mph	1.609	km/s	kilometers per second
Temperature Equations				

$$^{\circ}\text{C} = 5/9[\text{T}(^{\circ}\text{F}) - 32]$$

$$^{\circ}\text{F} = 9/5[\text{T}(^{\circ}\text{C}) + 32]$$

$$\text{Kelvin} = \text{T}(^{\circ}\text{C}) + 273.15$$

*Tchobanoglous, George, Schroeder, Edward D., *Water Quality Characteristics: Modeling and Modifications*, 1st ed., © 1985. Reprinted and Electronically reproduced by permission of Pearson Education, Inc., Upper Saddle River, New Jersey.

Appendix C

Properties of Water

The principal physical properties of water are summarized in Table C-1. They are described briefly below.

SPECIFIC WEIGHT	The specific weight of a fluid, γ , is the gravitational attractive force acting on a unit volume of a fluid. In SI units, the specific weight is expressed as kilonewtons per cubic meter (kN/m^3). At normal temperatures, γ is 9.81 kN/m^3 (62.4 lb/ft^3).
DENSITY	The density of a fluid, ρ , is its mass per volume. For water, ρ is 1000 kg/m^3 (1.94 slug/ft^3) at 4°C . There is a slight decrease in density with increasing temperature. The relationship between γ , ρ , and the acceleration due to gravity g is $\gamma = \rho g$.

Table C-1. Physical Properties of Water*

Temperature (°C)	Density ρ (kg/m^3)	Dynamic Viscosity $\mu \times 10^3$ ($\text{N}\cdot\text{s/m}^2$)	Kinematic Viscosity $\nu \times 10^6$ (m^2/s)	Surface Tensions [†] σ (N/m)	Specific Weight γ (kN/m^3)	Vapor Pressure p_v (kN/m^2)
0	999.8	1.781	1.785	0.0765	9.805	0.61
5	1000.0	1.518	1.519	0.0749	9.807	0.87
10	999.7	1.307	1.306	0.0742	9.804	1.23
15	999.1	1.139	1.139	0.0735	9.798	1.70
20	998.2	1.002	1.003	0.0728	9.789	2.34
25	997.0	0.890	0.893	0.0720	9.777	3.17
30	995.7	0.798	0.800	0.0712	9.764	4.24
40	992.2	0.653	0.658	0.0696	9.730	7.38
50	988.0	0.547	0.553	0.0679	9.698	12.33
60	983.2	0.466	0.474	0.0662	9.642	19.92
70	977.8	0.404	0.413	0.0644	9.589	31.16
80	971.8	0.354	0.364	0.0626	9.530	47.34
90	965.3	0.315	0.326	0.0608	9.466	70.10
100	958.4	0.282	0.294	0.0589	9.399	101.33

*At atmospheric pressure.

[†]In contact with air.

For most practical purposes, liquids may be regarded as incompressible. The bulk modulus of elasticity E is given by

MODULUS OF ELASTICITY

$$E = \frac{\Delta p}{\Delta V/V},$$

where Δp is the increase in pressure that, when applied to a volume V , results in a decrease in volume ΔV . In SI units the modulus of elasticity is expressed as kilonewtons per square meter (kN/m^2). For water, E is approximately 2.150 kN/m^2 (44.9 lb/ft^2) at normal temperatures and pressures.

The viscosity of a fluid, μ , is a measure of its resistance to tangential or shear stress. In SI units the viscosity is expressed as newton-seconds per square meter ($\text{N}\cdot\text{s/m}^2$).

DYNAMIC VISCOSITY

In many problems concerning fluid motion, the viscosity appears with the density in the form μ/ρ , and it is convenient to use a single term ν , known as the kinematic viscosity. The kinematic viscosity of a liquid, expressed as square meters per second (m^2/s) in SI units, diminishes with increasing temperature.

KINEMATIC VISCOSITY

Surface tension of a fluid, σ , is the physical property that enables a drop of water to be held in suspension at a tap, a glass to be filled with liquid slightly above the brim and yet not spill, and a needle to float on the surface of a liquid. The surfacetension force across any imaginary line at a free surface is proportional to the length of the line and acts in a direction perpendicular to it. In SI units, surface tension is expressed as newtons per meter (N/m). There is a slight decrease in surface tension with increasing temperature.

SURFACE TENSION

Vapor pressure is the partial pressure exerted by gas phase molecules that are in dynamic equilibrium with a liquid or solid. This means that the rate of evaporation of the gas is equal to its rate of condensation. The pressure exerted by this gas, or vapor, is known as the vapor pressure p_v . In SI units the vapor pressure is expressed as kilopascals (kPa) or kilonewtons per square meter (kN/m^2). Vapor pressure increases with decreasing intermolecular forces and increasing temperature.

VAPOR PRESSURE

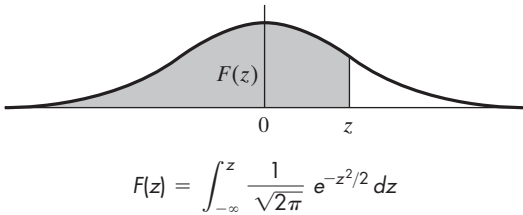
VENNARD, J. K., and R. L. STREET, 1975, *Elementary Fluid Mechanics*, 5th ed., John Wiley and Sons, New York.

REFERENCE

Appendix D

Normal Distribution Tables

Table D-1. Cumulative Normal Distribution*



z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.5000	.5040	.5080	.5120	.5160	.5199	.5239	.5279	.5319	.5359
.1	.5398	.5438	.5478	.5517	.5557	.5596	.5636	.5675	.5714	.5753
.2	.5793	.5832	.5871	.5910	.5948	.5987	.6026	.6064	.6103	.6141
.3	.6179	.6217	.6255	.6293	.6331	.6368	.6406	.6443	.6480	.6517
.4	.6554	.6591	.6628	.6664	.6700	.6736	.6772	.6808	.6844	.6879
.5	.6915	.6950	.6985	.7019	.7054	.7088	.7123	.7157	.7190	.7224
.6	.7257	.7291	.7324	.7357	.7389	.7422	.7454	.7486	.7517	.7549
.7	.7580	.7611	.7642	.7673	.7704	.7734	.7764	.7794	.7823	.7852
.8	.7881	.7910	.7939	.7967	.7995	.8023	.8051	.8078	.8106	.8133
.9	.8159	.8186	.8212	.8238	.8264	.8289	.8315	.8340	.8365	.8389
1.0	.8413	.8438	.8461	.8485	.8508	.8531	.8554	.8577	.8599	.8621
1.1	.8643	.8665	.8686	.8708	.8729	.8749	.8770	.8790	.8810	.8830
1.2	.8849	.8869	.8888	.8907	.8925	.8944	.8962	.8980	.8997	.9015
1.3	.9032	.9049	.9066	.9082	.9099	.9115	.9131	.9147	.9162	.9177
1.4	.9192	.9207	.9222	.9236	.9251	.9265	.9279	.9292	.9306	.9319
1.5	.9332	.9345	.9357	.9370	.9382	.9394	.9406	.9418	.9429	.9441
1.6	.9452	.9463	.9474	.9484	.9495	.9505	.9515	.9525	.9535	.9545
1.7	.9554	.9564	.9573	.9582	.9591	.9599	.9608	.9616	.9625	.9633
1.8	.9641	.9649	.9656	.9664	.9671	.9678	.9686	.9693	.9699	.9706
1.9	.9713	.9719	.9726	.9732	.9738	.9744	.9750	.9756	.9761	.9767
2.0	.9772	.9778	.9783	.9788	.9793	.9798	.9803	.9808	.9812	.9817
2.1	.9821	.9826	.9830	.9834	.9838	.9842	.9846	.9850	.9854	.9857
2.2	.9861	.9864	.9868	.9871	.9875	.9878	.9881	.9884	.9887	.9890
2.3	.9893	.9896	.9898	.9901	.9904	.9906	.9909	.9911	.9913	.9916
2.4	.9918	.9920	.9922	.9925	.9927	.9929	.9931	.9932	.9934	.9936
2.5	.9938	.9940	.9941	.9943	.9945	.9946	.9948	.9949	.9951	.9952
2.6	.9953	.9955	.9956	.9957	.9959	.9960	.9961	.9962	.9963	.9964
2.7	.9965	.9966	.9967	.9968	.9969	.9970	.9971	.9972	.9973	.9974

(continued)

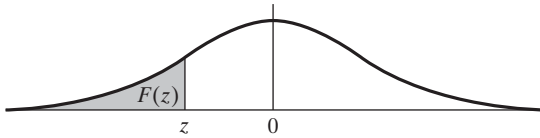
z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
2.8	.9974	.9975	.9976	.9977	.9977	.9978	.9979	.9979	.9980	.9981
2.9	.9981	.9982	.9982	.9983	.9984	.9984	.9985	.9985	.9986	.9986
3.0	.9987	.9987	.9987	.9988	.9988	.9989	.9989	.9989	.9990	.9990
3.1	.9990	.9991	.9991	.9991	.9992	.9992	.9992	.9992	.9993	.9993
3.2	.9993	.9993	.9994	.9994	.9994	.9994	.9994	.9995	.9995	.9995
3.3	.9995	.9995	.9995	.9996	.9996	.9996	.9996	.9996	.9996	.9997
3.4	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9998

*For more extensive tables, see National Bureau of Standards, *Tables of Normal Probability Functions*, Washington, D.C., U.S. Government Printing Office, 1953 (Applied Mathematics Series 23). Note that they show

$$\int_{-z}^z f(z) dz, \text{ not } \int_{-\infty}^z f(z) dz.$$

Source: E. L. Crow, F. A. Davis, and M. W. Maxfield, 1960, *Statistics Manual*, Dover Publications, New York, Table 1, p. 229.

Table D-2. Percentiles of the Normal Distribution*



$$F(z) = \int_{-\infty}^z \frac{1}{\sqrt{2\pi}} e^{-z^2/2} dz$$

F(z)	z	F(z)	z
.0001	-3.719	.500	.000
.0005	-3.291	.550	.126
.001	-3.090	.600	.253
.002	-2.878	.650	.385
.005	-2.576	.700	.524
.010	-2.326	.750	.674
.020	-2.054	.800	.842
.025	-1.960	.850	1.036
.040	-1.751	.900	1.282
.050	-1.645	.950	1.645
.100	-1.282	.960	1.751
.150	-1.036	.975	1.960
.200	-.842	.980	2.054
.250	-.674	.990	2.326
.300	-.524	.995	2.576
.350	-.385	.998	2.878
.400	-.253	.999	3.090
.450	-.126	.9995	3.291
.500	.000	.9999	3.719

*For a normally distributed variable x , we have $x = \mu + z\sigma$, where μ = mean of x and σ = standard deviation of x . For more extensive tables, see R. A. Fisher, and F. Yates, *Statistical Tables*, 4th rev. ed., Edinburgh, Oliver & Boyd, Ltd., 1953, pp. 39, 60-62.

Source: E. L. Crow, F. A. Davis, and M. W. Maxfield, 1960, *Statistics Manual*, Dover Publications, New York, Table 2, p. 230.

Appendix E

Useful Hydrology-Related Internet Links

TEXTBOOK <http://hydrology.rice.edu/>
the website contains presentation materials, detailed examples, cases, and datasets useful to both instructors and students of hydrology

This list of Web sites was prepared in January 2012, and Web addresses sometimes change. **For a current and updated list of these sites and active links, please visit the textbook website hydrology.rice.edu or www.pearson-highered.com/bedient.com/bedient.**

The reader should also note the availability of proprietary software for hydrologic analysis, which may be obtained from numerous commercial vendors. <http://www.vieuxinc.com/vflo.htm>

*Vflo*TM is a physics-based distributed model that preserves the spatial variation of slope, hydraulic roughness, infiltration, and rainfall input within a gridded representation of the watershed. The model uses a numerical solution of the kinematic wave equations to simulate overland and channel flow from rainfall runoff. Gridded parameters are derived from geospatial data and physical characteristics defining soil infiltration/moisture, channel and overland hydraulics, and rating curves. High-resolution rainfall rates from radar or rain gage sensors allow detailed predictions within the drainage network comprising the watershed. The desktop edition of *Vflo*TM is used to setup and calibrate the model for continuous or event-based simulation. In continuous simulation using long-term radar rainfall input covering days, months, or years, *Vflo*TM generates a time series of discharge, runoff volume, and soil moisture for watch points. The continuous model may be configured to output distributed maps of discharge and stage, or infiltration depths for recharge studies. Optional modules for *Vflo*TM provide functionality such as Inundation mapping, Automatic Calibration, Parameter Sensitivity, and spatially and temporally dynamic design storms. *Vflo*TM can be run in real-time for hydrologic prediction of discharge or stage/inundation depths. Real-time applications include flood warning and emergency notification.

WEATHER AND METEOROLOGY

<http://www.noaa.gov>

The National Oceanic and Atmospheric Association's Web site, which contains useful weather and climate information.

<http://www.nws.noaa.gov>

The National Weather Service provides up-to-date and in-depth forecasts for across the country.

<http://www.intellicast.com>

Intellicast is a commercial weather site that best features a Java-enabled NEXRAD radar loop of the nation.

<http://www.hpc.ncep.noaa.gov/tropical/rain/katrina2005.html>

The National Oceanic and Atmospheric Association's brief summary of Hurricane Katrina can be found here.

[http://ww2010.atmos.uiuc.edu/\(Gh\)/home.rxml](http://ww2010.atmos.uiuc.edu/(Gh)/home.rxml)

The weather world 2010 project, developed by the University of Illinois Urbana-Champaign, integrates current and archived weather data with multimedia instructional resources

<http://cirrus.sprl.umich.edu/wxnet/>

This University of Michigan site provides extensive weather information and contains links to about 300 other weather-related sites.

<http://www.ncdc.noaa.gov/oa/ncdc.html>

The National Climatic Data Center has a huge weather data archive available to the public through this site.

http://www.weatheroffice.ec.gc.ca/canada_e.html

This Canadian Meteorological Centre site provides access to statistical summaries, current conditions, and data archives across Canada.

<http://weather.unisys.com>

Unisys's site provides forecast data that is designed for the weather professional.

<http://www.weatherbug.com>

WeatherBug is a free program that gives the user real-time weather information and alerts while online.

<http://www.usbr.gov/pn/agrimet/>

AgriMet is a satellite-based network of automated agricultural weather stations operated by the U.S. Bureau of Reclamation that focuses mainly on the Pacific Northwest and Nevada.

<http://data.geocomm.com>

GIS DATA

The GIS Data Depot has large amounts of GIS data available for free download.

<http://water.usgs.gov/maps.html>

The USGS provides GIS data for watersheds. Perhaps most useful is its index of HUCs (Hydrologic Unit Codes), which are the numeric addresses for watersheds used by the USGS.

<http://www.fws.gov/nwi>

The U.S. Fish and Wildlife Service provides GIS data specific to wetlands.

<http://www.vieuxinc.com>

Vieux Inc. performs mathematical analysis on NEXRAD radar, determines actual amounts of rainfall over a watershed, and places the data in a format to be used in GIS.

<http://www.gis.com>

General site explaining what GIS is and some of its capabilities.

<http://www.esri.com>

ESRI is the developer of ArcView, one of the most popular GIS viewers available.

<http://www.census.gov/ftp/pub/geo/www/tiger>

The TIGER/Line files are a digital database of geographic features throughout the U.S. and pertinent information, such as their latitude and longitude.

<http://www.colorado.edu/geography/gcraft/notes/notes.html>

Notes and study materials for GIS and the geographer's craft.

<http://www.lsrp.com>

This site makes Remote Sensing algorithms, code, and technology available to all interested parties.

<http://www.pierssen.com/arcgis/arcgis.htm>

Avenue scripts, extensions, and other goodies.

<http://www.nwrc.usgs.gov/data.htm>

National Wetlands Research Center spatial data and metadata server.

<http://nhd.usgs.gov/index.html>

The National Hydrography Dataset (NHD) is a comprehensive set of digital spatial data that contains information about surface water features such as lakes, ponds, streams, rivers, springs, and wells.

<http://www.epa.gov/waterscience/BASINS>

Better Assessment Science Integrating Point and Nonpoint Sources (BASINS) integrates GIS, national watershed data, and state-of-the-art environmental assessment and modeling tools into one convenient package.

**GENERAL
HYDROLOGY**

<http://www.epa.gov/surf>

This U.S. Environmental Protection Agency site allows the user to find watershed maps, water quality information, and water use data.

<http://water.usgs.gov>

This portion of the USGS Web site provides real-time and historic stream flows, gage heights and GIS data.

<http://water.usgs.gov/nwis>

This is the primary location within the USGS Web site for retrieval of national water data.

<http://www.hec.usace.army.mil>

The U.S. Army Corps of Engineers' Hydrologic Engineering Center contains numerous modeling programs available for download, including HEC-HMS and HEC-RAS.

<http://www.usbr.gov>

The U.S. Bureau of Reclamation provides information on hundreds of dams and water-related facilities throughout the American West.

<http://www.weather.gov/oh>

The National Weather Service's Office of Hydrologic Development contains current stream information as well as information on past flood events.

<http://www.nrcs.usda.gov>

The Natural Resource Conservation Service, a branch of the U.S. Department of Agriculture, offers information on soils, wetlands, drought conditions, etc.

http://www.ems-i.com/GMS/GMS_Overview/gms_overview.html

This site explains the Groundwater Modeling System and consolidates a number of free groundwater models.

<http://www.dodson-hydro.com>

This site is maintained by consultants who provide support for a variety of hydrologic tools, including GIS, HEC-HMS, and HEC-RAS.

http://www.wmo.int/pages/prog/hwrrp/index_en.html

The World Meteorological Organization contains many international links to water-related activities.

<http://chl.erdc.usace.army.mil/>

The U.S. Army Corps of Engineers Waterways Experiment Station (WES) at Vicksburg, Mississippi, performs research and provides hydraulic models on many aspects of riverine, coastal, and environmental systems.

<http://glovis.usgs.gov>

Most recent Landsat 7 images from all over the world.

<http://www.computationalhydraulics.com/>

The Web site of Computational Hydraulics International provides links, publications, software, and other products related to stormwater and urban hydrology. CHI maintains very valuable list-serves for EPANET (a water distribution model), HEC models, HSPF, SWMM, and WASP (a receiving water quality model).

**URBAN
HYDROLOGY
LINKS**

<http://www.bmpdatabase.org>

This database provides access to best management practice (BMP) performance data in a standardized format for approximately 100 BMP studies conducted since the mid-1980s. The database was developed by the Urban Water Resources Research Council (UWRRC) under a cooperative agreement with the U.S. Environmental Protection Agency.

<http://www.stormwatercenter.net>

The Stormwater Manager's Resource Center provides technical assistance on stormwater management issues.

<http://www.stormwaterauthority.org>

Although this Web site has a Florida focus, the Stormwater News site provides many links to related Web sites, publications, conferences, and ongoing stormwater-related activities.

**WATERSHEDS
AND THE
ENVIRONMENT**

<http://www.epa.gov/waters>

Watershed Assessment, Tracking and Environmental Results (WATERS) is a tool that links several databases to provide watershed information in a mapp02ased format.

<http://www.ewrinstitute.org>

The Environmental and Water Resources Institute (EWRI) of the American Society of Civil Engineers (ASCE) is an organization devoted to helping professionals in their water and environmental-related careers.

**LOCAL AND
REGIONAL
HYDROLOGY
LINKS**

Many Web sites exist that address the weather or hydrology of a specific region as opposed to nationwide. A few representative sites have been listed here. While they may not be pertinent to the area the reader is in, they often contain links to other geographical regions and are interesting sites in and of themselves.

<http://www.hcoem.org/>

The Harris County Office of Emergency Management serves Houston, Texas. Rainfall data can be plotted from various rain gages, and stream flow and gage height data can be accessed as well.

<http://atlas.lsu.edu>

Louisiana State University's Atlas project provides GIS data for all of the state of Louisiana.

<http://www.sfwmd.gov/site/index.php?id=1>

The South Florida Water Management District includes a massive amount of information about the water and environment of South Florida and the Everglades.

<http://www.tnris.state.tx.us>

The Texas Natural Resources Information System (TNRIS), a division of the Texas Water Development Board, is the state's clearinghouse for natural resources data.

<http://www.srh.noaa.gov/abrfc>

Arkansas-Red Basin River Forecast Center, provided by NOAA and NWS.

<http://www.nwrfc.noaa.gov>

This Web site of the Northwest River Forecast Center of the National Oceanic and Atmospheric Administration provides current surface water conditions, including river stages and flows, in the Pacific Northwest.

<http://www.bpa.gov/corporate>

The Bonneville Power Administration provides extensive information on the Columbia River and its tributaries.

<http://www.nwd.usace.army.mil/home.asp>

The Northwest Division of the U.S. Army Corps of Engineers provides watershed data for the Columbia and Missouri River Basins.

<http://www.hec.usace.army.mil>

HEC-HMS, HEC-RAS, UNET, and other models developed at the Hydrologic Engineering Center of the U.S. Army Corps of Engineers can be downloaded or referenced from this primary Web site.

**DIRECT LINKS TO
FEDERAL MODELS
AND SOFTWARE**

http://water.usgs.gov/software/surface_water.html

This USGS site offers state-of-the-art statistical software, links to groundwater models, and software for flood frequency analysis, the HSPF model, and FEQ (Full Equations Model for river hydraulics).

<http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm>

Software for backwater analysis (WSPRO), culvert design, and other highway-related hydrology and hydraulics can be downloaded from this Web site of the Federal Highway Administration.

<http://www.epa.gov/ceampubl/>

The U.S. Environmental Protection Agency Center for Exposure Assessment Modeling (CEAM) distributes several models, including HSPF, SWMM (1994 version), and WASP, and provides software packages for analysis of environmental data.

<http://www.ccee.orst.edu/swmm>

The most current version of EPA SWMM is available at this Oregon State University Web site, though it is not an “official” EPA release.

<http://www.fema.gov/plan/prevent/fhm/index.shtm>

Software related to floodplain definition is provided or linked as part of the Flood Hazard Mapping activity of the Federal Emergency Management Agency’s National Flood Insurance Program.

Glossary

- activities** GIS data set that geographically describes human activities
- adiabatic lapse rate** rate of temperature change with elevation in the atmosphere
- adverse slope** channel slope that slopes upward in the downstream direction
- albedo** reflection coefficient
- alluvium** sediment deposited by flowing rivers and consisting of sands and gravel
- alto** middle level clouds, altocumulus or altostratus
- annual exceedances** series of n largest independent events in an n -yr period
- antecedent precipitation index** measure of the soil moisture
- aquiclude** relatively impermeable layer that does not allow water to flow through
- aquifer** geologic formation that is saturated and that transmits large quantities of water
- aquitard** low permeability layer that allows leakage to occur
- area-elevation curve** curve relating surface area to a particular elevation
- arithmetic mean** simple method for averaging rainfall
- artesian aquifer** confined aquifer that is under pressure greater than atmospheric pressure
- artesian well** well that penetrates a confined aquifer where the water level rises above the upper surface of the confined aquifer
- atmospheric pressure** force per unit area that air exerts on a surface
- auger hole test** field test for determining hydraulic conductivity by measuring a change in water level in a bore hole after a rapid addition or removal of a volume of water
- backward difference** finite difference evaluated by looking backward in space or time from a given point
- backwater curve** plot of water depth along the channel length

barometer instrument that measures atmospheric pressure

base flow flow in a channel due to soil moisture or ground water

best management practices (BMP) database of the effectiveness of storm-water management practices maintained by ASCE

Bias a systematic as opposed to a random distortion of a statistic as a result of sampling procedure

binomial coefficient the number of combinations of choosing x events out of n possible events

bored well well constructed by means of a hand-operated or power-driven auger

bulk density ratio of the oven-dried mass of a sample to its original volume

capillary rise height to which water will rise under capillary forces

capillary suction negative head in soil above the water table that holds water due to surface tension

capillary water water held above the water table due to capillary forces, caused by attraction of soil and water

Catchment sometimes used synonymously for just the surface portion of the watershed

central difference finite difference that is centered in space or time around a point of interest

central moments statistical moments with respect to the mean

Channel slope reflects the rate of change of elevation with distance along the main channel or within an overland flow area

characteristic curves a set of curves of x or y versus t , produced by the method of characteristics

characteristic method method of solving a set of equations by converting partial differential equations into ordinary differential equations

cirrus feathery or fibrous clouds

class interval a range into which data may be grouped

class mark midpoint of the class interval

coalescence process process by which droplets of rain increase in size through collision with other droplets

collectively exhaustive term describing events that account for all possible outcomes

collector channels small channels that collect overland flow and carry it to larger channels

combined sewer system of conduits that carries storm runoff and domestic sewage together

Complete Series when all of the data in a historic time series is used

computational block a set of several lines of data that describe one type of computation

condensation phase change of water vapor into liquid droplets

cone of depression plot of a surface of decline of the water table or piezometric level near a pumping well

confidence limits control curves between which a known percentage of data points is expected to fall

confined aquifer aquifer that is overlain by a confining unit consisting of a lower hydraulic conductivity, with potential water level above the confining unit

conformal projection coordinate projection that maintains local angles and shapes

constant head permeameter laboratory device that determines hydraulic conductivity by supplying a continuous source of water at a constant head

continuous simulation model model based on long-term water balance equations

continuous variable a variable that can assume any value on the real axis

contour (elevation) line connecting points on a surface having the same elevation

control section cross section of a channel that has a controlling structure (bridge, free outfall, or weir)

convective motion of air due to intense heating at ground level

conveyance measure of amount of flow carried in a channel, defined from Manning's equation, related to n , A , and P

convolution equation equation used to derive a storm hydrograph from a unit hydrograph (add and lag method)

Courant condition limiting condition for a numerical time step used to avoid stability problems

crest segment portion of the hydrograph that contains the peak flow

critical depth depth of water for which specific energy is minimum

critical flow flow of water through a channel for which the specific energy is minimum

critical slope channel slope for which uniform flow is critical, or $y = y_c$

cumulative density function (CDF) probability of nonexceedance as a function of a continuous random variable

cumulative distribution same as cumulative density function, but for either continuous or discrete variables

cumulative mass curve graph of total accumulated rainfall or other variable versus time

cumulus cloud with individual domes or towers, usually dense and well defined

cyclonic counterclockwise motion of air due to movement of large air mass systems (in Northern Hemisphere)

datum geographical reference frame, usually based on a vertical benchmark (mean sea level)

deep well water well dug to a large depth from the surface

degree-day type of equation used to determine snowmelt

density mass per unit volume of a substance

depression storage area of relatively lower elevation that stores precipitation or water

derived distribution statistical approach that derives the frequency distribution of runoff from the frequency distribution of rainfall

detention storage reservoir that detains water for a given time and then discharges entirely downstream

deterministic model in which parameters are based on physical relations for dynamic processes of hydrologic cycle

dew point temperature temperature at which the air just becomes saturated when cooled

diffusion model flood routing model based on solving a diffusion equation

digital elevation model (DEM) an array of numbers representing spatial distribution of elevations for a specific area, in digital form

digital terrain model (DTM) digital representation of terrain, including point data, contours, triangular facets, or a DEM

dimensionless hydrograph a general hydrograph developed from many unit hydrographs, used in the Soil Conservation Service method

Direct Runoff The remaining portion of precipitation that becomes overland flow; flows generally in a down-gradient direction to accumulate in local streams that then flow to rivers

direct runoff hydrograph graph of direct runoff (rainfall—losses) versus time

directly connected impervious area (DCIA) area that drains directly into a drainage system

discrete probability probability of a discrete event, such as rainfall occurrence

discrete variable variable that can assume only discrete values

distributed parameter model that attempts to describe physical processes and mechanisms in space and time

diversion structure that moves flood runoff to another area or watershed

drainage divides boundary that separates subbasin areas according to direction of runoff

drawdown curve plot of a curve of decline of the water table or piezometric level near a pumping well

driven well well constructed by driving a series of pipe lengths into the ground

dug well well excavated by hand

Dupuit parabola parabolic shape of the water table determined by the Dupuit equation

duration length of time during which rain falls

Dynamic Wave a model that uses the full unsteady non-uniform version of the momentum equation, is only used when both pressure and inertial forces are important

easterlies trade winds, located between 30° north latitude and the equator

effective flow area portion of the total cross-sectional area where flow velocity is normal to the cross section

effluent stream flow out of an aquifer into a stream during flood conditions

El Niño-Southern Oscillation (ENSO) a phenomenon, characterized by a warm phase associated with high sea surface temperatures off the coast of Peru, and strong winds aloft over the tropical Atlantic (creating high vertical wind shear and unfavorable conditions for hurricane development)

equal area projection coordinate projection that maintains area

equidistant projection coordinate projection that maintains distance

equilibrium discharge constant maximum outflow reached for a constant rainfall intensity that falls continuously

equipotential lines lines of equal piezometric or water table level used in the analysis of ground water flow

evaporation phase change of liquid water to water vapor

evapotranspiration the combined process of evaporation and transpiration through vegetation

event type of model that simulates a single storm response

explicit method method of solving a set of equations based on previously known data

extreme value family of distributions for hydrologic variables

falling head permeameter laboratory device that determines hydraulic conductivity by measuring the rate of fall of water in a column

features items that are placed on a map, part of GIS data structure

field capacity amount of water held in soil after gravitational water is drained

finite difference method used to solve differential equations by approximating them as algebraic terms over a grid

fixed format structured arrangement of data on a line with fixed column fields

flood hazard zone area that will flood with a given probability, located on a FEMA map

flood plain low lands adjoining a channel, river, stream, watercourse, or lake that have been or may be inundated by flood water

flood plain management an overall decision-making process whose goal is to achieve the best and most appropriate use of the floodplain

flood routing analyzing the movement of a flood wave through a river system

flood zoning nonstructural flood control alternative involving zoning laws in a floodplain

flood-proofing nonstructural flood control alternative involving protecting individual buildings from rising water

floodway area in the floodplain that contains the fastest moving water, found by modeling encroachment on either side of the channel such that the banks increase by 1 ft

floodway encroachment area that separates the floodway from the limits of the floodplain

flow duration curve plot of magnitude versus percent of time the magnitude is equaled or exceeded

flow line common path of a fluid particle, also called a streamline, in ground water

flow net set of orthogonal streamlines and equipotential lines used in hydrogeologic site investigations

flowing artesian well well that penetrates a confined aquifer where the water level rises above the ground surface

force main a pressurized conduit

forward difference finite difference evaluated by looking forward in space or time from a given point

free format a line of data that is not in a structured arrangement but is separated by commas or spaces

frequency the number of observations of a random variable

friction slope equal to the total energy slope in open channels and calculated with Manning's equation for gradually varied flow

front boundary between one air mass and another, usually associated with cold air

Froude number dimensionless parameter to characterize open channel flow

geographic information system (GIS) computer application that displays numerous types of spatial data (such as land use, soil type, or topography) and links that data with a map

geomorphology study of parameters that describe the physical nature of a watershed

goal seek Microsoft Excel function that allows one to vary the value in one specific cell until a formula that is dependent on that cell returns the desired results

gradually varied flow open channel flow that changes gradually so that one dimensional analysis applies in each reach

grate inlet structure used to cover an inlet to a sewer system to keep out debris

gravel pack material used near a well screen to stabilize the aquifer and provide an annular zone of high permeability

gravitational water water that will drain through the soil under the force of gravity

Groundwater water that flows in porous media in the subsurface in either shallow or deeper aquifer systems that can be pumped for water supply to agricultural and municipal water systems

Grouped Data Data which have been arranged in groups or classes rather than showing all the original figures, can be used in a histogram

grout seals material used to seal a well so that water is pumped only from a predetermined layer

histogram bar graph of data

horizontal slope channel slope that is horizontal

humidity measure of the relative amount of water vapor in the atmosphere

hurricane intense cyclonic storm that usually forms over tropical oceans

hydraulic conductivity ratio of velocity to hydraulic gradient, indicating permeability of porous media

hydraulic jump sudden transition from supercritical flow to subcritical flow

hydraulic routing flood routing method using both the continuity equation and the momentum equation

hydraulically connected impervious area impervious area that drains directly into the drainage system

hydrograph graph of discharge versus time

hydrologic cycle the continuous process of water movement near the earth's surface

hydrologic routing flood routing method that uses the continuity equation and a storage equation

hyetograph graph of rainfall intensity versus time

hygroscopic water moisture that remains adsorbed to the surface of soil grains in the unsaturated zone

ice crystal process process by which water vapor condenses onto frozen nuclei to form ice crystals

implicit method method of solving simultaneous equations over a finite difference grid system at each time step

independent term describing an event that does not influence the occurrence of another event

ineffective flow area portion of the total cross-sectional area where flow velocity is not in a downstream direction

infiltration movement of water from the surface into the soil

infiltration capacity rate at which water can enter soil with excess water on surface

influent stream flow into an aquifer from a stream during drought conditions

instantaneous unit hydrograph hydrograph produced by a unit rainfall falling for a duration D , when D approaches zero

intensity-duration-frequency curve statistical plot relating intensity, duration, and frequency of design rainfalls

interceptor conduit at downstream end of combined sewer that carries sewage to a treatment plant

interflow flow of water through the upper soil zones to a stream

intersection the occurrence of both of two events

intrinsic permeability property that indicates the ability of a geological unit to transmit water; a property of the porous media only

invert bottom elevation of a channel or pipe

isohyetal method method for areally weighting rainfall using contours of equal rainfall (isohyets)

isotropic condition in which hydraulic properties of an aquifer are equal in all directions

jet stream narrow band of high-speed winds at upper elevations

jettied well well constructed by directing a high-velocity stream of water downward

kinematic wave wave that has a unique function relating Q and y

L-moments (L for “linear”) are linear combinations of the ordered (ranked) samples of a variable

La Niña A cold phase that has low sea surface temperatures in the eastern Pacific and the opposite pressure and wind anomalies

lag time time from the center of mass of rainfall to the peak of the hydrograph

lapse rate rate of change of temperature with elevation in the atmosphere

latent heat amount of heat that must be removed or added for a phase change to take place

leaky aquifer an aquifer confined by a low permeability unit able to transmit water as recharge to an adjacent aquifer

left overbank floodplain area above the channel on the left-hand side of a cross section, looking downstream, in HEC-2 or HEC-RAS model

linear reservoir reservoir that has a linear relationship between storage and outflow

lognormal distribution for which the log of the random variable is distributed normally

loss rates loss of precipitation due to evaporation or infiltration in a hydrologic system

low flow flow at a depth below the crown of a culvert or lowest point of the low chord of a bridge

lumped parameter model model that aggregates spatially distributed parameters

main channels large channels that carry flow from collector channels to some outlet such as a lake or bay

major drainage system a group of major streams that carry runoff from large storms when the minor drainage system is full

median the middle value in a set of values arranged in ascending or descending order

method of characteristics see *characteristic method*

method of maximum likelihood statistical method for fitting distributions to data

method of moments method to fit a distribution using estimates of the mean, variance, and skewness

mild slope channel slope for which uniform flow is subcritical

minor drainage system system that carries runoff from small storms

mixed population two or more causation mechanisms for time series data

mode value at which the probability density function is a maximum; also the value that has the highest frequency within a range of values

model calibration process of parameter estimation using known data

model verification process of comparing parameter estimates against a new set of data once model has been calibrated

moisture content ratio of volume of water to total volume of a unit of porous media

monoclonal flood wave simple flood wave consisting of a step increase in discharge that moves downstream

Muskingum method method of river routing that uses the continuity equation and a linear storage relationship

mutually exclusive term describing events where the occurrence of one event “excludes” the possibility of the other event; two events are mutually exclusive if it is not possible for both of them to occur

native scale the scale or resolution at which data are actually collected

NEXRAD Acronym for Next Generation Radar. NEXRAD (WSR 88D) is an advanced system that employs the Doppler effect to analyze atmospheric and rainfall conditions.

nimbus dense, dark rain-bearing cloud

nonstationary varying with time

nonuniform flow flow of water through a channel that gradually changes with distance

normal bridge method bridge modeling in HEC-2 for low flow through culverts, bridges without piers, etc.

normal depth depth at which water will flow under uniform conditions

open channel flow flow of water through an open channel with a free water surface

optimum channel cross section cross section that requires a minimum flow area

orographic rainfall due to mechanical lifting of air over mountains

orthophoto photograph that has been rectified to a georeferenced coordinate system

outliers data points that fall an “unusually large” distance from the cumulative density function

overland flow flow of water across the land surface in a downgradient direction

overland flow planes area over which runoff flows in thin sheets

pan coefficient coefficient relating evaporation from a standard pan to evaporation from a lake

parameter optimization selection of the best value for a parameter

perched water table an unconfined aquifer situated on top of a confining unit separated from a main aquifer

permeability the rate a fluid passes through a porous medium

piezometers small-diameter well that measures water table or confined aquifer pressure

piezometric surface indicator of pressure or head level of water in an aquifer

plotting position empirical estimate of frequency or return period used for graphing data

population complete series of random events that belong to a given probability density function or probability mass function

porosity ratio of volume of voids to total volume of sample

porous medium geologic material that will allow water to flow through it

potential (initial) abstraction initial loss of water prior to infiltration in the SCS method of runoff analysis

potential evapotranspiration amount of water that could be lost to evapotranspiration if water supply was unlimited

potentiometric surface same as *piezometric surface*

precipitation water that falls to the earth in the form of rain, snow, hail, or sleet

pressure flow flow in a conduit with no free surface

prism storage prism-shaped volume of storage in a river reach

probability relative number of occurrences of an event after a large number of trials

probability density function (PDF) distribution of probability for a continuous random variable; integral is probability

Probability mass function (PMF) distribution of probability for a discrete random variable; sum is probability

probable maximum flood flood that results from PMP event

probable maximum precipitation (PMP) highest precipitation likely to occur under known meteorological conditions

projection transformation from a curved geographic coordinate system to a Cartesian coordinate system

psychrometer instrument that measures the relative humidity by measuring the difference in a thermometer with a wet bulb and a thermometer with a dry bulb

Modified Puls method a method of flood routing

pump device used to raise water to a higher level or to discharge water under pressure

pump test field test for determining hydraulic characteristics by pumping water from one well and observing the decline in water level in other wells

quantized-continuous continuous data presented in a discrete form due to the measurement process

radar device used to measure distances to objects by measuring the reflection of electromagnetic radiation emitted by a transmitter

random variable parameter that cannot be predicted with certainty

rapidly varied flow flow of water through a channel whose characteristics are rapidly changing

raster data spatially continuous information that is usually collected on a regularly spaced grid

rating curve relationship between depth and amount of flow in a channel

rational method simple linear rainfall-runoff relationship that predicts peak outflow

recession curve portion of the hydrograph where runoff is from base flow

recharge water that infiltrates to an aquifer, usually from rainfall

recurrence interval time interval in which an event will occur once on the average

reflectivity measure of the reflected radar signal, usually measured in decibels (dBZ)

regional skew coefficient skew parameter based on several different gage analyses in a region

regulator hydraulic control structure that diverts combined sewage into an interceptor

relative frequency ratio of number of observations in a class interval to total number

relative humidity ratio of water vapor pressure to the saturated vapor pressure at the same temperature

Reliability-1 risk

reservoir man-made storage area for flood control of water supply

retention storage reservoir or pond that retains some flood water without allowing it to discharge downstream

return period time interval for which an event will occur once on the average

right overbank floodplain area above the channel on the right-hand side of a cross section, looking downstream

rising limb portion of the hydrograph where runoff is increasing

risk probability that an event may occur

river routing flood routing where outflow depends on both river and flood-plain storage

root zone depth to which the major vegetation draws water through a root system in soil

routing parameter coefficient that accounts for attenuation of the peak flow through a channel segment

Runge-Kutta methods numerical scheme for solving ordinary differential equations

runoff coefficient ratio of runoff to precipitation

runoff curve number parameter used in the SCS method that accounts for soil type and land use

Saint Venant equations pair of continuity and momentum equations that govern hydraulic flood routing

saturated thickness thickness of an aquifer measured from the water table or the confining layer to the bottom

saturated vapor pressure partial pressure of water vapor when air is saturated (no more evaporation can occur)

saturated zone zone of an aquifer in which the porous media is under pressure greater than atmospheric pressure

S-curve method method for converting a unit hydrograph of one duration to a new duration

separated sewer system of conduits that carry storm runoff and domestic sewage separately

shallow well well dug less than a few meters deep

side-flow weir weir discharging to side of flow direction

simulation model model describing the reaction of a watershed to a storm using numerical equations

skew coefficient skewness divided by cube of standard deviation

skewness third central moment about the mean, a measure of asymmetry

slug test field test for determining hydraulic conductivity by injecting or removing a volume of water in a single well and observing the decline or recovery of water level

snow water equivalent (SWE) depth of water that would result if all the snow melted

soil moisture storage volume of water held in the soil

soil water zone portion of soil from the ground surface through the root zone

special bridge method modeling in HEC-2 or HEC-RAS for bridges with pier structures or where pressure or weir flow occurs

specific energy the sum of elevation head and velocity head at a cross section referenced to the channel bed

specific yield volume of water released from unconfined aquifer per unit area per unit drop in head

spheroid geometric solid produced by rotating an ellipse about its short axis

squalls strong line of storms, often preceding a front

standard deviation measure of spread about the mean for a data set

standard step method numerical backwater computation that determines water surface elevations for an open channel

station estimate skewness estimate using data from a single station

steep slope channel slope for which uniform flow is supercritical

stereographic projection projection of the earth onto a plane tangent to the North Pole

stochastic model that contains a random component

storage coefficient volume of water that an aquifer releases from or takes into storage per unit surface area per unit change in piezometric head

storage delay constant parameter that accounts for lagging of the peak flow through a channel segment

storage indication curve relationship among storage, outflow, and Δt

storage routing flood routing in which outflow is uniquely determined by the amount of storage

storage-discharge relation values that relate storage in the system to outflow from the system

storage-indication method reservoir routing that uses the continuity equation and a storage-indication curve

storm sewer system of conduits that carries storm runoff

stratus low, gray cloud layer where precipitation is mostly drizzle

stream function mathematical description of the flow lines for a given set of equipotential lines, associated with a flow net

streamlines flow lines that indicate the direction of water movement in a flow net

subbasins hydrologic divisions of a watershed that are relatively-homogeneous

subcritical flow of water at a velocity less than critical, or tranquil flow

subsurface stormflow flow of water through the upper soil zones to a stream

supercritical flow of water at a velocity greater than critical, or rapid flow

surcharge condition in which the water level in a storm sewer rises above the crown of the conduit, or pressure flow

synthetic design storm rainfall hyetograph obtained through statistical means

synthetic unit hydrograph unit hydrograph for ungaged basins based on theoretical or empirical methods

Thiessen method method for areally weighting rainfall through graphical means

tide gate structure installed at the outlet of a sewer system or channel to prevent backflow

time base total duration of direct runoff under the hydrograph

time of concentration time at which outflow from a basin is equal to inflow or time of equilibrium

time of rise time from the start of rainfall excess to the peak of the hydrograph

time series analysis statistical method to evaluate time variable data

time to peak time from the center of mass of rainfall to the peak of the hydrograph

time-area histogram bar graph of translation time versus incremental area

time-area method simple rainfall-runoff relationship based on the concept of pure translation of runoff

total energy the sum of the pressure head, elevation head, and velocity head at a cross section

tracer test injection and observation of a chemical into an aquifer to measure rate and direction of ground water flow

transform method hydrologic method to convert rainfall to runoff

transient flow flow characterized by time-variable flow

transmissivity measure of how easily water in a confined aquifer can flow through the porous media, $T = kb$, where k = hydraulic conductivity, b = thickness

transpiration conversion of water to water vapor through plant tissue

trapezoidal rule standard method of numerical integration

Tropical Cyclones see hurricanes

type curve used in Theis analysis of a pumping well in a confined aquifer

unconfined aquifer aquifer that does not have a confining unit and is defined by a water table

uniform flow open channel prismatic flow for which slope of total energy equals bottom slope

uniformity coefficient measure of the uniformity of grain sizes in a soil sample

union set of all events that are members of event A or of event B, or both

unit hydrograph graph of runoff versus time produced by a unit rainfall for a given duration over a watershed

vadose zone zone of aeration that extends from the surface to the water table including the capillary fringe

vapor pressure partial pressure exerted by water vapor

variance second central moment about the mean, a measure of scale or width

vector data data that maps geographic features using lines drawn between points or coordinates

velocity head energy due to the velocity of water

velocity potential product of total head and hydraulic conductivity, the gradient of which defines velocity from Darcy's law

water divide point where the water table is at a maximum and flow does not cross

water surface profile plot of the depth of water in a channel along the length of the channel

water table the surface where fluid pressure is exactly atmospheric or the depth to which water will rise in a well screened in an unconfined aquifer

water well vertical hole with casing and screen designed to pump water

watershed area of land that drains to a single outlet and is separated from other watersheds by a divide

watershed divide line or border that defines a watershed topographically

Watershed relief the elevation difference between two reference points within a watershed

Watershed slope see channel slope.

wave celerity speed at which a wave moves downstream in a channel

wave cyclone low level circulation developing along a front

wedge storage wedge-shaped volume of storage in a river reach that accounts for rising and falling stages

weir flow flow at a depth greater than the top of a roadway of a bridge; flow over the top of a channel structure

well vertical hole dug into the soil that penetrates an aquifer and is usually cased and screened

well casing lining that prevents entry of unwanted water into a well

well screen slotted casing that allows water to enter a well from the aquifer that it penetrates

westerlies westerly winds in the middle latitudes

wilting point moisture content below which plants cannot extract further water

ϕ index simple infiltration method that assumes that loss is uniform across the rainfall pattern

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