# CHAPTER 1

# INTRODUCTION

1-1 **PURPOSE**. This document establishes general concepts and procedures for the hydrologic design of surface structures for the U.S. Army, Navy, Air Force, Marine Corps, and Federal Aviation Administration (FAA).

1-2 **SCOPE**. This manual prescribes the hydrologic design criteria to be used for transportation facilities and other areas.

1-3 **REFERENCES**. Appendix A contains a list of references used in this UFC. Appendix D is a bibliography that lists publications that are considered relevant to this subject and that offer additional information on various topics.

1-4 **UNITS OF MEASUREMENT**. The unit of measurement system in this document is the inch-pound (IP). In some cases, International System of Units (SI) measurements may be the governing critical values because of applicable codes, accepted standards, industry practices, or other considerations.

1-5 **APPLICABILITY**. Criteria in this manual pertain to all Department of Defense (DOD) military facilities in the United States, it territories, trusts, and possessions, and unless otherwise noted, to DOD facilities overseas on which the United States has vested base rights. For DOD facilities overseas, if written agreement exists between host nation and DOD that requires application of either North Atlantic Treaty Organization(NATO), International Civil Aviation Organization (ICAO), or other standards, those standards shall apply as stipulated in the agreement.

1-5.1 **Previous Standards.** The criteria in this manual are not intended to apply to existing facilities constructed under previous standards; however, when existing facilities are modified or new facilities are constructed, they must conform to the criteria established in this manual unless waived.

1-5.2 **Applicability Within DOD.** This document covers a wide range of topics in the areas of surface drainage and serves as the standard for several agencies responsible for hydrologic design for transportation facilities and other areas. The intended use of the facility under design may differ between agencies and in some cases dictates the need for separate standards. In special cases in which more than one standard is presented, or the standard does not apply to all agencies, special care has been given to clearly identify the relevant audience. Any user of this manual should pay close attention to the relevance of each topic to the intended agency.

1-5.3 **Design Objectives**

1-5.3.1 The objective of storm drainage design is to provide for safe passage of vehicles or operation of the facility during the design storm event. The drainage system is designed to collect storm water runoff from the pavement surface and adjacent areas, convey it along and through the adjacent areas, and discharge it to an adequate receiving body without causing adverse on- or off-site impacts.

1-5.3.2 Storm water collection systems must be designed to provide adequate surface drainage. Traffic safety is intimately related to surface drainage. Rapid removal of storm water from the pavement minimizes the conditions which can result in the hazards of hydroplaning. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity.

1-5.3.3 The objective of storm water conveyance systems (e.g., storm drain piping, ditches and channels, pumps) is to provide an efficient mechanism for conveying design flows from inlet locations to the discharge point without surcharging inlets or otherwise causing surface flooding. Erosion potential must also be considered in the design of open channels or ditches used for storm water conveyance.

1-5.3.4 The design of appropriate discharge facilities for storm water collection and conveyance systems includes consideration of storm water quantity and quality. Local, state, and/or Federal regulations often control the allowable quantity and quality of storm water discharges. To meet these regulatory requirements, storm drainage systems will usually require detention or retention basins, and/or other best management practices (BMPs) for the control of discharge quantity and quality.

1-5.4 **Waivers to Criteria.** Each DOD service component is responsible for setting administrative procedures necessary to process and grant formal waivers. Waivers to the criteria contained in this manual shall be in accordance with Appendix E.

1-6 **GENERAL INVESTIGATIONS**. An on-site investigation of the system site and tributary area is a prerequisite for study of drainage requirements. Information regarding capacity, elevations, and condition of existing drains will be obtained. Topography, size and shape of drainage area, and extent and type of development; profiles, cross sections, and roughness data on pertinent existing streams and watercourses; and location of possible ponding areas will be determined. Thorough knowledge of climatic conditions and precipitation characteristics is essential. Adequate information regarding soil conditions, including types, permeability, vegetative cover, depth to and movement of subsurface water, and depth of frost will be secured. Outfall and downstream flow conditions, including high-water occurrences and frequencies, also must be determined. The effect of base drainage construction on local interests' facilities and local requirements that will affect the design of the drainage works will be evaluated. Where diversion of runoff is proposed, particular effort will be made to avoid resultant downstream conditions leading to unfavorable public relations, costly litigations, or damage claims. Any agreements needed to obtain drainage easements and/or avoid interference with water rights will be determined at the time of design and consummated prior to initiation of construction. Possible adverse effects on water quality due to disposal of drainage in waterways involved in water supply systems will be evaluated.

## 1-7 ENVIRONMENTAL CONSIDERATIONS

1-7.1 **National Environmental Policy**. The National Environmental Policy Act of 1969 (NEPA), approved 1 January 1970, sets forth the policy of the Federal Government, in cooperation with state and local governments and other concerned public and private organizations, to protect and restore environmental quality. The Act (Public Law [PL] 91-190) states, in part, that Federal agencies have a continuing responsibility to use all practicable means, consistent with other essential considerations of national policy, to create and maintain conditions under which man and nature can exist in productive harmony. Federal plans, functions, and programs are to be improved and coordinated to (1) preserve the environment for future generations, (2) assure safe, healthful, productive, and aesthetically pleasing surroundings for all, (3) attain the widest beneficial uses of the environment without degradation, risk to health or safety or other undesirable consequences, …and (4) enhance the quality of renewable resources and approach the maximum attainable recycling of depletable resources. All Federal agencies, in response to NEPA, must be concerned not just with the impact of their activities on technical and economic considerations but also on the environment.

1-7.2 **Federal Guidelines.** Storm drainage design is an integral component in the design of transportation facilities. Drainage design for transportation facilities must strive to maintain compatibility and minimize interference with existing drainage patterns, control flooding of the pavement surface for design flood events, and minimize potential environmental impacts from facility-related storm water runoff. To meet these goals, the planning and coordination of storm drainage systems must begin in the early planning phases of transportation projects. Federal goals for sustainability are outlined in the Environmental Protection Agency's (EPA) *Federal Guide for Green Construction Specs*.

System planning, prior to commencement of design, is essential to the successful development of a final storm drainage design. Successful system planning will result in a final system design that evolves smoothly through the preliminary and final design stages of the transportation project.

1-7.3 **Regulatory Considerations.** The regulatory environment related to drainage design is ever changing and continues to grow in complexity. Engineers responsible for the planning and design of drainage facilities must be familiar with Federal, state, and local regulations, laws, and ordinances that may impact the design of storm drain systems. A detailed discussion of the legal aspects of highway drainage design, including a thorough review of applicable laws and regulations, is included in the American Association of State Highway and Transportation Officials' (AASHTO) *Highway Drainage Guidelines*, Volume V, and *Model Drainage Manual*, Chapter 2. Some of the more significant Federal, state, and local regulations affecting drainage design are summarized in paragraphs 1-7.4 through 1-7.6.

1-7.4 **Federal Regulations.** The following Federal laws may affect the design of storm drainage systems. The highway drainage engineer should be familiar with these laws and any associated regulatory procedures.

1-7.4.1 The Fish and Wildlife Act of 1956 (*Title 16 United States Code* [USC] Section 742a, et seq.), the Migratory Game-Fish Act (16 USC § 760c-760g), and the Fish and Wildlife Coordination Act (16 USC § 661-666c) express the concern of Congress with the quality of the aquatic environment as it affects the conservation, improvement and enjoyment of fish and wildlife resources. The Fish and Wildlife Service's role in the permit review process is to review and comment on the effects of a proposal on fish and wildlife resources. Storm drainage design may impact streams or other channels which fall under the authority of these acts.

1-7.4.2 NEPA (42 USC § 4321-4347) declares the national policy to promote efforts which will prevent or eliminate damage to the environment and biosphere, stimulate the health and welfare of man, and to enrich the understanding of the ecological systems and natural resources important to the nation. NEPA and its implementing guidelines from the Council on Environmental Quality and the Federal Highway Administration (FHWA) affect highway drainage design as it relates to impacts on water quality and ecological systems.

1-7.4.3 Section 401 of the Federal Water Pollution Control Act Amendments of 1972 (FWPCA) (PL 92-500, 86 Stat. 816, 33 USC § 1344) prohibits discharges from point sources unless covered by a National Pollutant Discharge Elimination System (NPDES) permit. These permits are issued under authority of Section 402 of the Act, and must include the more stringent of either technology-based standards or water-quality based standards. The NPDES program regulations are found at Title 40, Code of Federal Regulations, Parts 122-125 (40 CFR 122-125). These regulations govern how the EPA and authorized states write NPDES permits by outlining procedures on how permits shall be issued, what conditions are to be included, and how the permits should be enforced.

1-7.4.4 Section 402p of the FWPCA (PL 92-500, 86 Stat. 816, 33 USC § 1344) requires the EPA to establish final regulations governing storm water discharge permit application requirements under the NPDES program. The permit application requirements include storm water discharges associated with industrial activities. Highway construction and maintenance are classified as industrial activities.

1-7.4.5 The Water Quality Act of 1987 (PL 100-4), an amendment of Section 402p of the FWPCA, provides a comprehensive framework for the EPA to develop a phased approach to regulating storm water discharges under the NPDES program for storm water discharges associated with industrial activity (including construction activities). The Act clarified that permits for discharges of storm water associated with industrial activity must meet all of the applicable provisions of Section 402 and Section 301, including technology and water quality-based standards. The classes of diffuse sources of pollution include urban runoff, construction activities, separate storm drains, waste disposal activities, and resource extraction operations, which all correlate well with categories of discharges covered by the NPDES storm water program.

1-7.4.6 Section 404 of the FWPCA (PL 92-500, 86 Stat. 816, 33 USC § 1344) prohibits the unauthorized discharge of dredged or fill material in navigable waters. The instrument of authorization is termed a permit, and the Secretary of the Army, acting through the Chief of Engineers, U.S. Army Corps of Engineers, has responsibility for the administration of the regulatory program. The definition of navigable waters includes all coastal waters, navigable waters of the United States to their headwaters, streams tributary to navigable waters of the United States to their headwaters, inland lakes used for recreation or other purposes that may be interstate in nature, and wetlands contiguous or adjacent to the above waters. A water quality certification is also required for these activities.

1-7.4.7 The Coastal Zone Management Act of 1972 (PL 92-583, amended by PL 94-310; 86 Stat. 1280, 16 USC § 145, et seq.) declares a national policy to preserve, protect, develop, and restore or enhance the resources of the nation's coastal zone, and to assist states in establishing management programs to achieve wise use of land and water resources, giving full consideration to ecological, cultural, historic, and aesthetic values as well as to the needs of economic development. The development of highway storm drainage systems in coastal areas must comply with this act in accordance with state coastal zone management programs.

1-7.4.8 The Coastal Zone Act Reauthorization Amendments of 1990 (CZARA) specifically charged state coastal programs (administered under Federal authority by the National Oceanic and Atmospheric Administration [NOAA]), and state nonpoint source programs (administered under Federal authority by the EPA), to address nonpoint source pollution issues affecting coastal water quality. The guidance specifies economically achievable management measures to control the addition of pollutants to coastal waters for sources of nonpoint pollution through the application of the best available nonpoint pollution control practices, technologies, processes, siting criteria, operating methods, or other alternatives.

1-7.4.9 The Safe Water Drinking Act of 1974, as amended, includes provisions for requiring protection of surface water discharges in areas designated as sole or principal source aquifers. Mitigation of activities that may contaminate the aquifer (including highway runoff) are typically required to assure Federal funding of the project, which may be withheld if harm to the aquifer occurs.

1-7.5 **State Regulations.** In addition to complying with the Federal laws cited in paragraphs 1-7.1 through 1-7.4.9, the design of storm drainage systems must also comply with state laws and regulations. State drainage law is derived from both common and statutory law. A summary of applicable state drainage laws originating from common law, or court-made law, and statutory law follow. Note that this is a generalized summary of common state drainage law. Drainage engineers should become familiar with the application of these legal principles in their states.

1-7.5.1 The civil law rule of natural drainage is based upon the perpetuation of natural drainage. A frequently quoted statement of this law is:

> . . .every landowner must bear the burden of receiving upon his land the surface water naturally falling upon land above it and naturally

> flowing to it therefrom, and he has the corresponding right to have the surface water naturally falling upon his land or naturally coming upon it, flow freely therefrom upon the lower land adjoining, as it would flow under natural conditions. From these rights and burdens, the principle follows that he has a lawful right to complain of others, who, by interfering with natural conditions, cause such surface water to be discharged in greater quantity or in a different manner upon his land, than would occur under natural conditions. . . . (Heier v. Krull. 160 Cal 441 (1911))

This rule is inherently strict, and as a result has been modified to some degree in many states.

1-7.5.2 The reasonable use rule states that the possessor of land incurs liability only when his harmful interference with the flow of surface waters is unreasonable. Under this rule, a possessor of land is legally privileged to make a reasonable use of his land even though the flow of surface waters is altered thereby and causes some harm to others. The possessor of land incurs liability, however, when his harmful interference with the flow of surface waters is unreasonable.

1-7.5.3 Stream water rules are founded on a common law maxim that states that "water runs and ought to run as it is by natural law accustomed to run." Thus, as a general rule, any interference with the flow of a natural watercourse to the damage of another will result in liability. Surface waters from highways are often discharged into the most convenient watercourse. The right is unquestioned if those waters were naturally tributary to the watercourse and unchallenged if the watercourse has adequate capacity; however, if all or part of the surface waters has been diverted from another watershed to a small watercourse, any lower owner may complain and recover for ensuing damage.

1-7.5.4 Eminent domain is a statutory law giving public agencies the right to take private property for public use. This right can be exercised as a means to acquire the right to discharge highway drainage across adjoining lands when this right may otherwise be restricted. Whenever the right of eminent domain is exercised, a requirement of just compensation for property taken or damaged must be met.

1-7.5.5 Agricultural drainage laws have been adopted in some states. These laws provide for the establishment, improvement, and maintenance of ditch systems. Drainage engineers may have to take into consideration agricultural laws that may or may not permit irrigation waste water to drain into the highway right-of-way. If the drainage of irrigated agricultural lands into roadside ditches is permitted, excess irrigation water may have to be provided for in the design of the highway drainage system.

1-7.5.6 Environmental quality acts have been enacted by many states to promote the enhancement and maintenance of the quality of life. Hydraulic engineers should be familiar with these statutes.

1-7.6 **Local Laws.** Many governmental subdivisions have adopted ordinances and codes that impact drainage design. These include regulations for erosion control, BMPs, and storm water detention.

1-7.6.1 Erosion control regulations set forth practices, procedures, and objectives for controlling erosion from construction sites. Cities, counties, or other government subdivisions commonly have erosion control manuals that provide guidance for meeting local requirements. Erosion control measures are typically installed to control erosion during construction periods, and are often designed to function as a part of the highway drainage system. Additionally, erosion control practices may be required by the regulations governing storm water discharge requirements under the NPDES program. These erosion and sediment control ordinances set forth enforceable practices, procedures, and objectives for developers and contractors to control sedimentation and erosion by setting specific requirements that may include adherence to limits of clearing and grading, time limit or seasonal requirements for construction activities to take place, stabilization of the soil, and structural measures around the perimeter of the construction site.

1-7.6.2 BMP regulations set forth practices, procedures, and objectives for controlling storm water quality in urbanizing areas. Many urban city or county government bodies have implemented BMP design procedures and standards as a part of their land development regulations. The design and implementation of appropriate BMPs for controlling storm water runoff quality in these areas must be a part of the overall design of highway storm drainage systems. Additionally, the NPDES permit program for storm water management addresses construction site runoff by the use of self-designed storm water pollution prevention plans. These plans are based upon three main types of BMPs: those that prevent erosion, others that prevent the mixing of pollutants from the construction site with storm water, and those that trap pollutants before they can be discharged. All three of these BMPs are designed to prevent, or at least control, the pollution of storm water before it has a chance to affect receiving streams.

1-7.6.3 Storm water detention regulations set forth practices, procedures, and objectives for controlling storm water quantity through the use of detention basins or other controlling facilities. The purpose of these facilities is to limit increases in the amount of runoff resulting from land development activities. In some areas, detention facilities may be required as a part of the highway storm drainage system. Detention and retention basins must generally meet design criteria to control the more frequent storms and to safely pass larger storm events. Storm water management may also include other measures to reduce the rate of runoff from a developed site, such as maximizing the amount of runoff that infiltrates back into the ground.

1-7.7 **U.S. Army Environmental Quality Program.** Army Regulation (AR) 200-1, outlines the Army's fundamental environmental policies, management of its programs, and its various types of activities, one of which, water resources management, includes minimizing soil erosion and attendant pollution caused by rapid runoff into streams and rivers. The overall goal is to "plan, initiate, and carry out all actions and programs in a manner that will minimize or avoid adverse effects on the quality of the human environment without impairment of the Army mission." A primary objective is to eliminate the discharge of pollutants produced by Army activities. Provision of suitable surface drainage facilities is necessary in meeting this objective.

1-7.8 **U.S. Air Force Environmental Quality Program**. Air Force policy directive (AFPD) 32-70 enunciates Air Force policy in compliance with NEPA executive orders and DOD directives. Procedures outlined in AFPD 32-70 are similar to those described for Army installations. Air Force instruction (AFI) 32-7061 establishes 32 CFR 989 as the controlling document on environmental assessments and statements for Air Force facilities.

1-7.9 **U.S. Navy Environmental Quality Program**. The Navy's Environmental Quality Initiative (EQI) is a comprehensive initiative focused on maximizing the use of pollution prevention to achieve and maintain compliance with environmental regulations. The EQI is a fundamental part of the Navy environmental strategy called AIMM to SCORE – Assess, Implement, Manage and Measure to achieve Sustained Compliance and Operational Readiness through Environmental Excellence.

1-7.10 **FAA Environmental Handbook**. FAA Order 5050.4 provides instructions and guidance for preparing and processing the environmental assessments, findings of no significant impact (FONSI), and environmental impact statements (EIS) for airport development proposals and other airport actions as required by various laws and regulations.

1-7.11 **Environmental Impact Analysis**. A comprehensive reference, *Handbook for Environmental Impact Analysis*, was issued in September of 1974. This document, prepared by the U.S. Army Corps of Engineers Construction Engineering Research Laboratory (CERL), presents recommended procedures for use by Army personnel in preparing and processing environmental impact assessments (EIA) and EIS. The procedures list step-by-step actions considered necessary to comply with requirements of NEPA and subsequent guidelines. These require that all Federal agencies use a systematic and disciplinary approach to incorporate environmental considerations into their decision making process.

1-7.12 **Environmental Effects of Surface Drainage Systems.** Such facilities in the arctic or subarctic could have either beneficial or adverse environmental impacts affecting water, land, ecology, and socioeconomic (human and economic) considerations. Despite low population density and minimal development, the fragile nature of the ecology in the arctic and subarctic has attracted the attention of environmental groups interested in protecting these unique assets. Effects on surrounding land and vegetation may cause changes in various conditions in the existing environment, such as surface water quantity and quality, groundwater levels and quality, drainage areas, animal and aquatic life, and land use. Proposed systems may also have social impacts on the community, requiring relocation of military and public activities, open space, recreational activities, community activities, and quality of life. Environmental attributes related to water could include such items as erosion, aquifer yield, flood potential, flow or temperature variations (the latter affecting permafrost levels and ice jams), biochemical oxygen demand, and content of dissolved oxygen, dissolved solids, nutrients, and coliform organisms. These are among many possible attributes to be considered in evaluating environmental impacts, both beneficial and adverse, including effects on surface water and groundwater. Various methods are explained for presenting and summing up the impact of these effects on the environment.

1-7.13 **Discharge Permits**. The Federal pollution abatement program requires regulatory permits for all discharges of pollutants from point sources (such as pipelines, channels, or ditches) into navigable waters or their tributaries. This requirement does *not* extend to discharges from separate storm sewers except where the storm sewers receive industrial, municipal, and agricultural wastes or runoff, or where the storm water discharge has been identified as a significant contributor of pollution by the EPA regional administrator, the state water pollution control agency, or an interstate agency. Federal installations, while cooperating with and furnishing information to state agencies, do not apply for or secure state permits for discharges into navigable waters.

1-7.14 **Effects of Drainage Facilities on Fish.** In many locations, natural drainage channels are environmentally important to preserve fish resources. Culverts, ditches, and other drainage structures constructed along or tributary to these fish streams must be designed to minimize adverse environmental effects. Culvert hazards to fish include high inverts, excessive velocities, undersized culverts, stream degradation, failed or damaged culverts that create obstructions, erosion and siltation at outlets, blockage by icing, and seasonal timing and methods of drainage construction. Consult Federal and state fish and wildlife agencies for guidance on probable effects and possible expedients to mitigate them. Give special concern to anticipated conditions during fish migration season. More information is located in Chapter 4.

# CHAPTER 2

# SURFACE HYDROLOGY

2-1 **PURPOSE AND SCOPE.** This chapter presents explanations and examples to give a better understanding of problems in the design of drainage facilities, and outlines convenient methods of estimating design capacities for drainage facilities.

2-2 **HYDROLOGIC CRITERIA.** The Rational Method, developed over 100 years ago, is widely used for estimating design runoff from urban areas. The Rational Formula, popular because of its simplicity in application, is suited mainly to sizing culverts, storm drains, or channels to accommodate drainage from small areas, generally less than 200 acres. Selection of appropriate values of runoff coefficients in the formula depends on the experience of the designers and the designers' knowledge of local rainfall-runoff relationships. United States Geological Survey (USGS) regression equations and National Resources Conservation Service (NRCS) techniques appropriate for surface drainage design are also included in this chapter.

2-2.1 **Design Objectives.** The design capacity of surface drainage systems should economically drain the facilities with due consideration of the mission and importance of the particular facility and environmental impacts.

2-2.2 **Degree of Drainage Required.** The degree of protection to be provided by the drain system depends largely on the importance of the facility as determined by the type and volume of traffic to be accommodated, the necessity for uninterrupted service, and similar factors. Although the degree of protection should increase with the importance of the facility, minimum requirements must be adequate to avoid hazards to operation. One severe accident chargeable to inadequate drainage can offset any difference between the cost of reasonably adequate and inadequate drainage facilities. In some cases, one can justify use of design storm frequencies appreciably higher than minimum criteria in order to protect important facilities. In some designs, portions of the drainage system have been based on as high as a 50-year (yr) design frequency to reduce likelihood of flooding a facility essential to operations and to prevent loss of life.

2-2.3 **Surface Runoff from Design Storm.** Surface runoff from the selected design storm will be disposed of without damage to facilities, undue saturation of the subsoil, or significant interruption of normal traffic. In addition, certain facilities may have restrictions on surface storage of water due to the potential attraction of waterfowl. For more information on waterfowl hazards, refer to Air Force pamphlet (AFPAM) 91-212 or Advisory Circular (AC) 150/5200-33.

2-2.4 **Design Storm Frequency**

2-2.4.1 **DOD Airfields and Heliports.** For airfields and heliports, a minimum of a 2-yr storm event is required unless a waiver is obtained. This event shall have no encroachment of runoff on taxiway and runway pavements (including paved shoulders).

It should be noted that after this design storm frequency is specified, computations must be made to determine the critical duration of rainfall required to produce the maximum rate of runoff for each area. This will depend primarily on the slope and length of overland flow. Another important aspect of the design is minimizing ponding during rain events. Ponding is the accumulation of water around an inlet structure during a rain event. Typically, ponding will be limited around the apron inlets such that it does not exceed 4 inches (in.).

2-2.4.2 **Federal Aviation Administration.** For airports, it is recommended that the 5-yr storm event be used with no encroachment of runoff on taxiway and runway pavements (including paved shoulders). The damage or inconvenience that may be caused by storms greater than the 5-yr event may not warrant the increased cost of a drainage system large enough to accommodate that storm. The calculation of and provision for the storage of water or ponding between runways, taxiways, and aprons should usually be considered as a safety factor for temporary accommodation of runoff from storm return periods longer than 5 years. Ponding or storage of water of more than a temporary nature may be acceptable on the airport site other than between runways, taxiways, and aprons. Such temporary storage may indeed be essential because of limitations in offsite outfalls. An additional design consideration is the ponding of water around an inlet structure on an apron during a rain event. Typically, ponding will be limited around an apron inlet such that it does not exceed 4 inches.

2-2.4.3 **Areas Other Than Airfields.** For such developed portions of military installations as roadways, administrative, industrial, and housing areas, the design storm will normally be based on rainfall of 10-yr frequency. Potential damage or operational requirements may warrant a more severe criterion; in certain storage and recreational areas, a lesser criterion may be appropriate. (With concurrence of the using service, a lesser criterion may also be employed in regions where storms of an appreciable magnitude are infrequent and either damages or operational capabilities are such that large expenditures for drainage are not justified.)

An additional design consideration is the spread of water around inlets. Spread is the width of water on the paved surface measured perpendicular to the curb face. More information on limitations of spread can be found in Chapter 3.

2-2.5 **Surface Runoff from Storms Exceeding Design Storm.** The design storm frequency alone is not a reliable criterion of the adequacy of storm drain facilities. It is advisable to investigate the probable consequences of storms more severe and less frequent than the design storm before making final decisions regarding the adequacy of proposed drain-inlet capacities. Surface runoff from storms greater than the design storm will be disposed of with the minimum damage to the airfield or heliport. The center 50 percent of runways; the center 50 percent of taxiways serving these runways; and helipad surfaces along the centerline should be free from ponding resulting from storms of a 10-yr frequency and intensity determined by the geographic location. For areas other than airfields and heliports, check with the appropriate local regulatory agency for guidance on design storm requirements.

2-2.6 **Reliability of Operation.** The drainage system will have the maximum reliability of operation practicable under all conditions, with due consideration given to abnormal requirements such as debris and annual periods of snowmelt and ice jam breakup.

2-2.7 **Environmental Impact**. Drainage facilities will be constructed with minimal impact on the environment.

2-2.8 **Maintenance.** The drainage system will require minimum maintenance, and that maintenance will be accomplished quickly and economically. Particular reliance will be placed on maintenance of drainage components serving operational facilities.

2-2.9 **Future Expansion.** Future expansion of drainage facilities will be feasible with the minimum of expense and interruption to normal traffic.

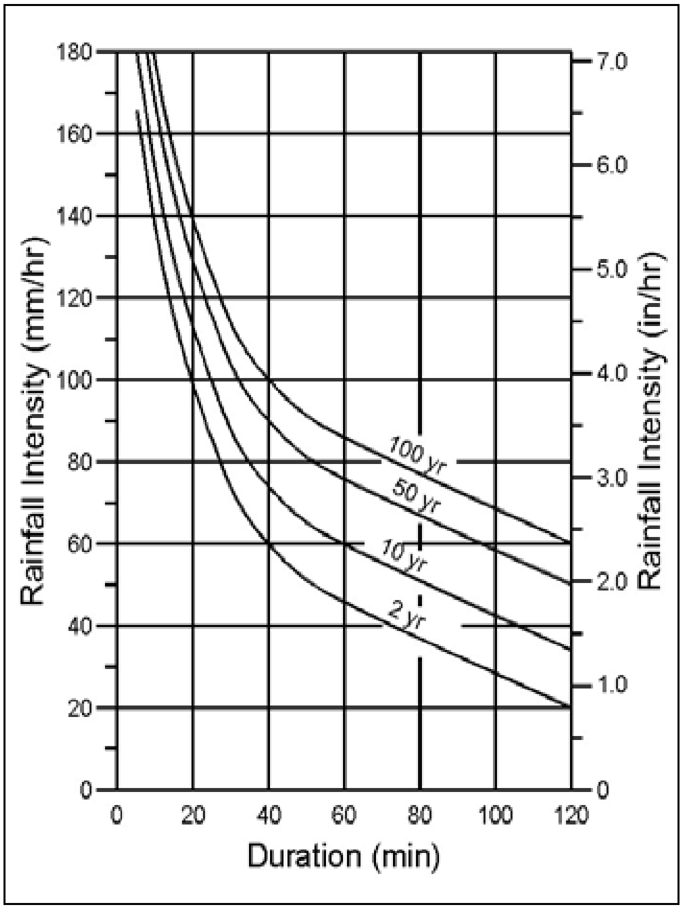
2-3 **HYDROLOGIC METHODS AND PROCEDURES.** This section provides an overview of hydrologic methods and procedures commonly used in drainage design. These methods include: the Rational Method, the Soil Conservation Service (SCS) Technical Release 55 (TR-55) method, and the USGS regression equations. Much of the information contained in this section was condensed from the FHWA Hydraulic Engineering Circular No. 22 (HEC-22). The presentation here is intended to provide the reader with an introduction to the methods and procedures, their data requirements, and their limitations. Most of these procedures can be applied using commonly available computer programs. Chapter 12 of this manual contains information on available computer programs.

2-3.1 **Rainfall (Precipitation).** Rainfall, along with watershed characteristics, determines the flood flows upon which storm drainage design is based. In this section, we will describe the constant rainfall and the synthetic rainfall techniques.

2-3.1.1 **Constant Rainfall Intensity.** Although rainfall intensity varies during precipitation events, many of the procedures used to derive peak flow are based on an assumed constant rainfall intensity. Intensity is defined as the rate of rainfall and is typically given in units of inches per hour (in/hr).

Intensity-duration-frequency curves (IDF curves) have been developed for many jurisdictions throughout the United States through frequency analysis of rainfall events for thousands of rainfall gages. The IDF curve provides a summary of a site's rainfall characteristics by relating storm duration and exceedance probability (frequency) to rainfall intensity (assumed constant over the duration). Figure 2-1 illustrates an example IDF curve. To interpret an IDF curve, find the rainfall duration along the X-axis, go vertically up the graph until reaching the proper return period, then go horizontally to the left and read the intensity off of the Y-axis. Regional IDF curves are available in most state or local highway agency drainage manuals. If the IDF curves are not available, the designer needs to develop them on a project-by-project basis.

**Figure 2-1. Example IDF Curve**




2-3.1.2 **Synthetic Rainfall Events.** Drainage design is usually based on synthetic rather than actual rainfall events. The SCS 24-hour (hr) rainfall distributions are the most widely used synthetic hyetographs. These rainfall distributions were developed by the U.S. Department of Agriculture SCS, which is now known as NRCS. The SCS 24-hr distributions incorporate the intensity-duration relationship for the design return period. This approach is based on the assumption that the maximum rainfall for any duration within the 24-hr duration should have the same return period. For example, a 10-yr, 24-hr design storm would contain the 10-yr rainfall depths for all durations up to 24 hours as derived from IDF curves. SCS developed four synthetic 24-hr rainfall distributions as shown in Figure 2-2; approximate geographic boundaries for each storm distribution are shown in Figure 2-3.

**Figure 2-2. SCS 24-hr Rainfall Distribution**

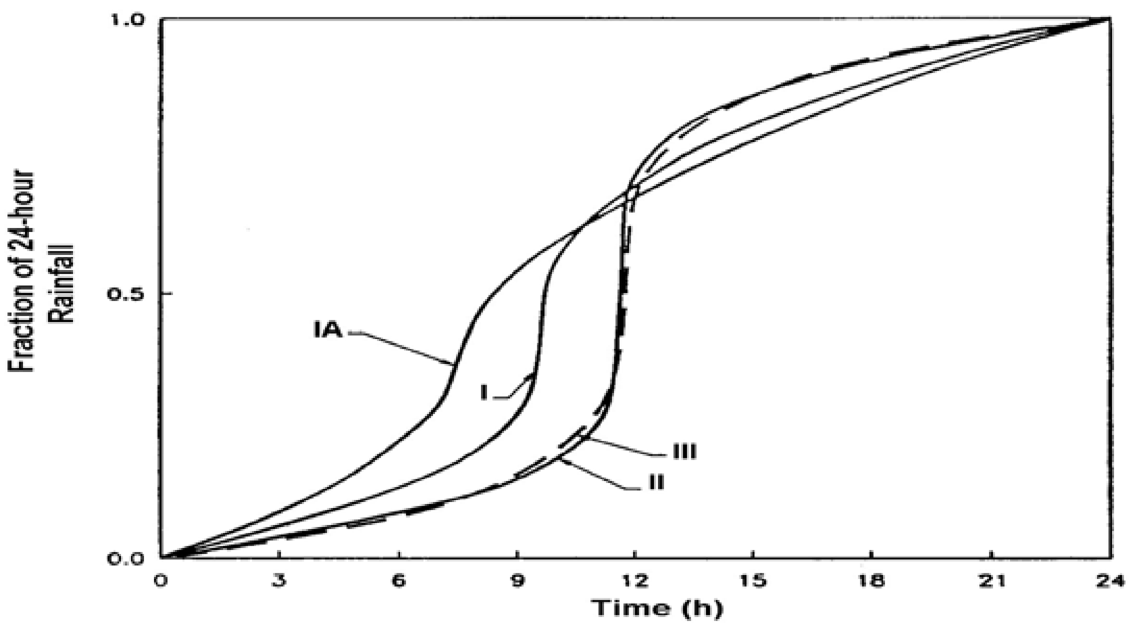
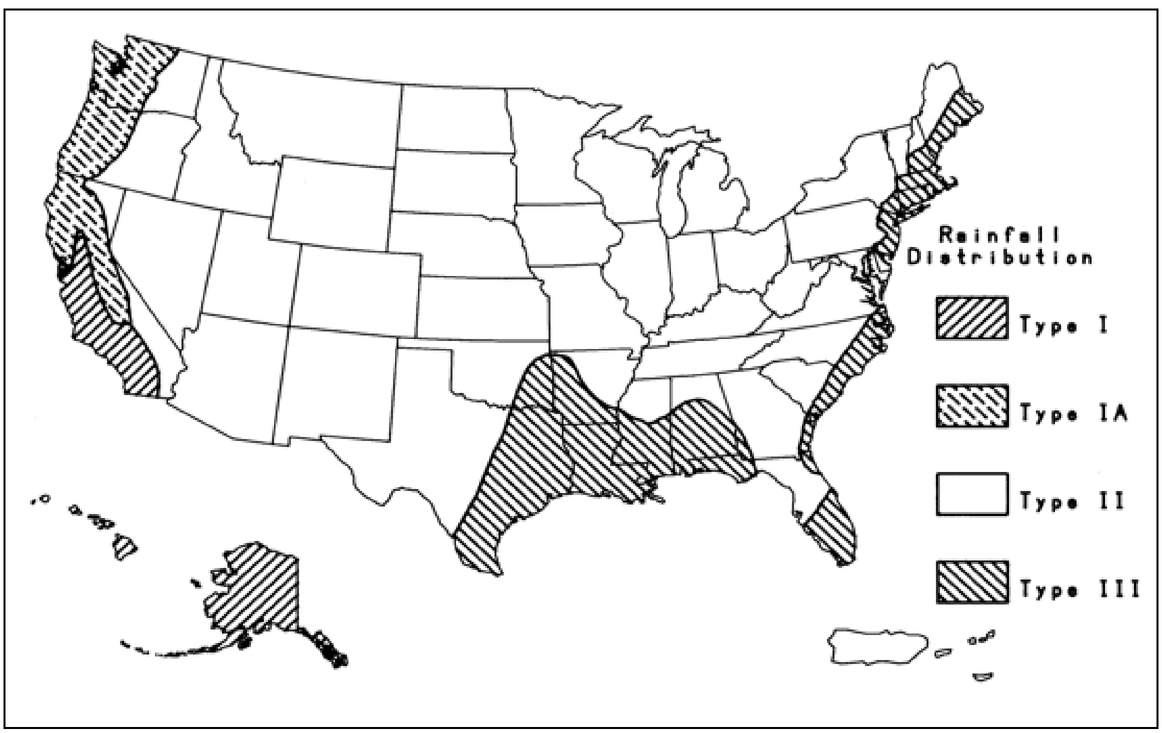



**Figure 2-3. Approximate Geographic Areas for SCS Rainfall Distributions**




Although the SCS distributions shown do not agree exactly with IDF curves for all locations in the region for which they are intended, the differences are within the accuracy limits of the rainfall depths from the Weather Bureau's rainfall frequency atlases.

2-3.2 **Determination of Peak Flow Rates.** Peak flows are generally adequate for design and analysis of conveyance systems such as storm drains or open channels; however, if the design or analysis must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is required. This section discusses three methods, the Rational Method, the SCS TR-55 method, and the USGS regression equations, that are used to derive peak flows for both gaged and ungaged sites. Each method can be used to develop a peak discharge. The drainage area of the project usually dictates which of these methods should be used. The Rational Method is the most commonly used method, but due to its assumptions, it is limited to drainage areas smaller than 200 acres. For drainage areas up to 2000 acres, the SCS TR-55 method is commonly used. Due to the way in which the regression equations were developed, they are usually not appropriate for very small areas, but each set of equations has its own limitations and those should be understood before the equations are applied. The regression equations are often used to compute the discharges for larger areas such as those necessary for culvert design.

2-3.2.1 **Rational Method.** One of the most commonly used equations for the calculation of peak flow from small areas is the Rational Formula, given as Equation 2-1:

$$Q = CIA \qquad (2\text{-}1)$$

where:

$Q$ = flow, $ft^3/s$

$C$ = dimensionless runoff coefficient representing the characteristics of the watershed

$I$ = rainfall intensity, in/hr

$A$ = drainage area, hectares, acres

2-3.2.1.1 **Assumptions**. Assumptions inherent in the Rational Formula are that:

- Peak flow occurs when the entire watershed is contributing to the flow.
- Rainfall intensity is the same over the entire drainage area.
- Rainfall intensity is uniform over a time duration equal to the time of concentration ($t_c$). The time of concentration is the time required for water to travel from the hydraulically most remote point of the basin to the point of interest.

- The frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 10-yr rainfall intensity is assumed to produce the 10-yr peak flow.

- The coefficient of runoff is the same for all storms of all recurrence probabilities.

2-3.2.1.2 **Limitations**. Because of the inherent assumptions, the Rational Formula should be applied only to drainage areas smaller than 200 acres.

### 2-3.2.2 Runoff Coefficient

2-3.2.2.1 The runoff coefficient, *C*, in Equation 2-1 is a function of the ground cover and a host of other hydrologic abstractions. It relates the estimated peak discharge to a theoretical maximum of 100 percent runoff. Typical values for *C* are given in Table 2-1. If the basin contains varying amounts of different land cover or other abstractions, a composite coefficient can be calculated through area weighing using Equation 2-2:

$$\text{weighted } C = \frac{\sum (C_x A_x)}{A_{total}} \qquad (2\text{-}2)$$

where:

$x$ = subscript designating values for incremental areas with consistent land cover

**Table 2-1. Runoff Coefficients for Rational Formula**

| Type of Drainage Area | Runoff Coefficient, *C** |
|---|---|
| Business: | |
| Downtown areas | 0.70 - 0.95 |
| Neighborhood areas | 0.50 - 0.70 |
| Residential: | |
| Single-family areas | 0.30 - 0.50 |
| Multi-units, detached | 0.40 - 0.60 |
| Multi-units, attached | 0.60 - 0.75 |
| Suburban | 0.25 - 0.40 |
| Apartment dwelling areas | 0.50 - 0.70 |
| Industrial: | |
| Light areas | 0.50 - 0.80 |
| Heavy areas | 0.60 - 0.90 |
| Parks, cemeteries | 0.10 - 0.25 |
| Playgrounds | 0.20 - 0.40 |
| Railroad yard areas | 0.20 - 0.40 |
| Unimproved areas | 0.10 - 0.30 |

| Type of Drainage Area | Runoff Coefficient, *C** |
|---|---|
| Lawns: | |
| Sandy soil, flat, 2 percent | 0.05 - 0.10 |
| Sandy soil, average, 2 to 7 percent | 0.10 - 0.15 |
| Sandy soil, steep, 7 percent | 0.15 - 0.20 |
| Heavy soil, flat, 2 percent | 0.13 - 0.17 |
| Heavy soil, average, 2 to 7 percent | 0.18 - 0.22 |
| Heavy soil, steep, 7 percent | 0.25 - 0.35 |
| Streets: | |
| Asphaltic | 0.70 - 0.95 |
| Concrete | 0.80 - 0.95 |
| Brick | 0.70 - 0.85 |
| Drives and walks | 0.75 - 0.85 |
| Roofs | 0.75 - 0.95 |
| *Higher values are usually appropriate for steeply sloped areas and longer return periods because infiltration and other losses have a proportionally smaller effect on runoff in these cases. | |

2-3.2.2.2 Example 2-1 illustrates the calculation of the runoff coefficient, *C*, using area weighing.

<u>Example 2-1</u>

*Given*: These existing and proposed land uses:

Existing conditions (unimproved):

| Land Use | Area, acres | Runoff Coefficient, *C* |
|---|---|---|
| Unimproved Grass | 22.1 | 0.25 |
| Grass | 21.2 | 0.22 |
| Total = | 43.3 | |

Proposed conditions (improved):

| Land Use | Area, acres | Runoff Coefficient, *C* |
|---|---|---|
| Paved | 5.4 | 0.90 |
| Lawn | 1.6 | 0.15 |
| Unimproved Grass | 18.6 | 0.25 |
| Grass | 17.7 | 0.22 |
| Total = | 43.3 | |

*Find*: Weighted runoff coefficient, *C*, for the existing and proposed conditions.

*Solution*:

Step 1. Determine weighted *C* for existing (unimproved) conditions using Equation 2-2.

$$\text{weighted } C = \frac{\sum (C_x A_x)}{A}$$

$$\text{weighted } C = \frac{[(22.1)(0.25) + (21.2)(0.22)]}{(43.3)}$$

$$\text{weighted } C = 0.235$$

Step 2. Determine weighted *C* for proposed (improved) conditions using Equation 2-2.

$$\text{weighted } C = \frac{[(5.4)(0.90) + (1.6)(0.15) + (18.6)(0.25) + (17.7)(0.22)]}{(43.3)}$$

$$\text{weighted } C = 0.315$$

2-3.2.3 **Rainfall Intensity.** Rainfall intensity, duration, and frequency curves are necessary to use the Rational Method. Regional IDF curves are available in most state and local highway agency manuals and are also available from NOAA. If the IDF curves are not available, they should be developed.

2-3.2.4 **Time of Concentration.** A number of methods can be used to estimate time of concentration, $t_c$, some of which are intended to calculate the flow velocity within individual segments of the flow path (e.g., shallow concentrated flow, open channel flow, etc.). The time of concentration can be calculated as the sum of the travel times within the various consecutive flow segments. For additional discussion on establishing the time of concentration for inlets and drainage systems, see Chapters 3 and 6 of this manual.

2-3.2.4.1 **Sheet Flow Travel Time**. Sheet flow is the shallow mass of runoff on a planar surface with a uniform depth across the sloping surface. This usually occurs at the headwater of streams over relatively short distances, rarely more than about 400 feet (ft), and possibly less than 80 ft. Sheet flow is commonly estimated with a version of the kinematic wave equation, a derivative of Manning's equation, shown as Equation 2-3:

$$T_{ti} = \frac{K_c}{I^{0.4}} \left( \frac{nL}{\sqrt{S}} \right)^{0.6} \qquad (2\text{-}3)$$

where:

$T_{ti}$ = sheet flow travel time, minutes (min)

$n$ = roughness coefficient (see Table 2-2)

$L$ = flow length, ft

$I$ = rainfall intensity, in/hr

$S$ = surface slope, feet per feet (ft/ft)

$K_c$ = empirical coefficient equal to 0.933

**Table 2-2. Manning's Roughness Coefficient (*n*) for Overland Sheet Flow**

| Surface Description | *n* |
|---|---|
| Smooth asphalt | 0.011 |
| Smooth concrete | 0.012 |
| Ordinary concrete lining | 0.013 |
| Good wood | 0.014 |
| Brick with cement mortar | 0.014 |
| Vitrified clay | 0.015 |
| Cast iron | 0.015 |
| Corrugated metal pipe | 0.024 |
| Cement rubble surface | 0.024 |
| Fallow (no residue) | 0.05 |
| Cultivated soils | |
| Residue cover < 20 percent | 0.06 |
| Residue cover > 20 percent | 0.17 |
| Range (natural) | 0.13 |
| Grass | |
| Short grass prairie | 0.15 |
| Dense grasses | 0.24 |
| Bermuda grass | 0.41 |
| Woods* | |
| Light underbrush | 0.40 |
| Dense underbrush | 0.80 |
| *When selecting *n*, consider cover to a height of about 1.2 inches. This is only part of the plant cover that will obstruct sheet flow. | |

Since the rainfall intensity value, $I$, depends on $t_{ti}$ and $t_{ti}$ is not initially known, the computation of $t_{ti}$ is an iterative process. An initial estimate of $t_{ti}$ is assumed and used to obtain $I$ from the IDF curve for the locality. The $t_{ti}$ is then computed from Equation 2-3 and used to check the initial value of $t_{ti}$. If they are not the same, the process is repeated until two successive $t_{ti}$ estimates are the same.

2-3.2.4.2 **Shallow Concentrated Flow Velocity**. After short distances of at most 300 ft, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using a relationship between velocity and slope as shown in Equation 2-4:

$$V = (3.28)kS_p^{0.5} \tag{2-4}$$

where:

$V$ = velocity, ft/s

$k$ = intercept coefficient (see Table 2-3)

$S_p$ = slope, percent

**Table 2-3. Intercept Coefficients for Velocity vs. Slope Relationship of Equation 2-4**

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

2-3.2.4.3 **Open Channel and Pipe Flow Velocity**. Flow in gullies empties into channels or pipes. Open channels are assumed to begin where either the blue stream line shows on USGS quadrangle sheets or the channel is visible on aerial photographs. Cross-section geometry and roughness should be obtained for all channel reaches in the watershed. Manning's equation can be used to estimate average flow velocities in pipes and open channels as follows:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{2-5}$$

where:

$n$ = roughness coefficient (see Table 2-4)

$V$ = velocity, ft/s

$R$ = hydraulic radius (defined as the flow area divided by the wetted perimeter), ft

$S$ = slope, ft/ft

**Table 2-4. Values of Manning's Coefficient (*n*) for Channels and Pipes**

| Conduit Material | Manning's *n** |
|---|---|
| Closed Conduits | |
| Brick | 0.013 - 0.017 |
| Cast iron pipe | |
| Cement-lined and seal coated | 0.011 - 0.015 |
| Concrete (monolithic) | 0.012 - 0.014 |
| Concrete pipe | 0.011 - 0.015 |
| Corrugated-metal pipe – 0.5 in. by 2.5 in. corrugations | |
| Plain | 0.022 - 0.026 |
| Paved invert | 0.018 - 0.022 |
| Spun asphalt lines | 0.011 - 0.015 |
| Plastic pipe (smooth) | 0.011 - 0.015 |
| Vitrified clay | |
| Pipes | 0.011 - 0.015 |
| Liner plates | 0.013 - 0.017 |
| Open Channels | |
| Lined channels | |
| Asphalt | 0.013 - 0.017 |
| Brick | 0.012 - 0.018 |
| Concrete | 0.011 - 0.020 |
| Rubble or riprap | 0.020 - 0.035 |
| Vegetal | 0.030 - 0.400 |
| Excavated or dredged | |
| Earth, straight and uniform | 0.020 - 0.030 |
| Earth, winding, fairly uniform | 0.025 - 0.040 |

| Conduit Material | Manning's *n** |
|---|---|
| Rock | 0.030 - 0.045 |
| Unmaintained | 0.050 - 0.140 |
| Natural channels (minor streams, top width at flood stage < 100 ft) | |
| Fairly regular section | 0.030 - 0.070 |
| Irregular section with pools | 0.040 - 0.100 |
| *Lower values are usually for well-constructed and maintained (smoother) pipes and channels. | |

For a circular pipe flowing full, the hydraulic radius is one-fourth of the diameter. For a wide rectangular channel ($W > 10\ d$), the hydraulic radius is approximately equal to the depth. The travel time is then calculated as follows:

$$T_{ti} = \frac{L}{60V} \tag{2-6}$$

where:

$T_{ti}$ = travel time for segment i, min

$L$ = flow length for segment i, ft

$V$ = velocity for segment i, ft/s

Example 2-2

*Given*: These flow path characteristics:

| Flow Segment | Length (ft) | Slope (ft/ft) | Segment Description |
|---|---|---|---|
| 1 (sheet flow) | 223 | 0.005 | Bermuda grass |
| 2 (shallow conduit) | 259 | 0.006 | Grassed waterway |
| 3 (flow in conduit) | 479 | 0.008 | 15-in concrete pipe |

*Find*: Time of concentration, $t_c$, for the area.

*Solution*:

Step 1. Calculate time of concentration for each segment.

**Segment 1**

Obtain Manning's *n* roughness coefficient from Table 2-2: $n = 0.41$

Determine the sheet flow travel time using Equation 2-3:

$$T_{ti} = \frac{K_c}{I^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6}$$

Since the rainfall intensity value, *I*, is being sought and is also in the equation, an iterative approach must be used. From experience, estimate a time of concentration and read a rainfall intensity from the appropriate IDF curve. In this example, try a time of concentration of 30 min and read from the IDF curve in Figure 2-1 an intensity of 3.4 in/hr. Now use Equation 2-3 to see how good the 30-min estimate was.

First, solve the equation in terms of *I*.

$$T_{ti1} = \left[\frac{0.933}{(I)^{0.4}}\right]\left[\frac{(0.41)(223)}{(0.005)^{0.5}}\right]^{0.6} = \frac{(68.68)}{I^{0.4}}$$

Inserting 3.4 in/hr for *I*, the result is 42.1 min. Since 42.1 is greater than the assumed 30 min, try the intensity for 42 min from Figure 2-1, which is 2.8 in/hr.

Using 2.8 in/hr, the result is 45.4 min. Repeat the process with 2.7 in/hr for 45 min and the result is a time of 46.2. This value is close to the 45.2 min.

Use 46 min for segment 1.

**Segment 2**

Obtain the intercept coefficient, *k*, from Table 2-3: *k* = 0.457 and $K_c$ = 3.281

Determine the concentrated flow velocity from Equation 2-4:

$$V = 3.28kS_p^{0.5} = (3.28)(0.457)(0.6)^{0.5} = 1.16 \text{ ft/s}$$

Determine the travel time from Equation 2-6:

$$T_{ti2} = \frac{L}{(60V)} = \frac{259}{[(60)(1.16)]} = 3.7 \text{ min}$$

**Segment 3**

Obtain Manning's *n* roughness coefficient from Table 2-4: $n = 0.011$

Determine the pipe flow velocity from Equation 2-5 (assuming full flow)

$$V = (1.49/0.011)(1.25/4)0.67\ (0.008)0.5 = 5.58 \text{ ft/s}$$

Determine the travel time from Equation 2-6:

$$T_{ti3} = \frac{L}{(60V)} = \frac{479}{[(60)(5.58)]} = 1.4 \text{ min}$$

Step 2. Determine the total travel time by summing the individual travel times:

$t_c = T_{ti1} + T_{ti2} + T_{ti3} = 46.0 + 3.7 + 1.4 = 51.1$ min   Use 51 min

<u>Example 2-3</u>

*Given*: Land use conditions from Example 2-1 and the following times of concentration:

| **Condition** | **Time of concentration $t_c$ (min)** | **Weighted *C* (from Example 2-1)** |
|---|---|---|
| Existing condition (unimproved) | 88 | 0.235 |
| Proposed condition (improved) | 66 | 0.315 |

Area = 43.36 acres

*Find*: The 10-yr peak flow using the Rational Formula and the IDF curve shown in Figure 2-1.

*Solution*:

Step 1. Determine the rainfall intensity, *I*, from the 10-yr IDF curve for each time of concentration.

Existing condition (unimproved) 1.9 in/hr

Proposed condition (improved) 2.3 in/hr

Step 2. Determine peak flow rate, *Q*.

Existing condition (unimproved):

$$Q = CIA$$

$$= (0.235)(1.9)(43.3)$$

$= 19.3 \text{ ft}^3/\text{s}$

Proposed condition (improved):

$Q = CIA$

$= (0.315)(2.3)(43.3)$

$= 31.4 \text{ ft}^3/\text{s}$

2-3.3 **USGS Regression Equations.** Regression equations are commonly used for estimating peak flows at ungaged sites or sites with limited data. The USGS has developed and compiled regional regression equations that are included in a computer program called the National Flood Frequency program (NFF). NFF allows quick and easy estimation of peak flows throughout the United States. All the USGS regression equations were developed using dependent variables in English units. Local equations may be available to provide better correspondence to local hydrology than the regional equations found in NFF. For more information on NFF, refer to paragraph 12-10.7.

2-3.3.1 **Rural Equations.** The rural equations are based on watershed and climatic characteristics within specific regions of each state that can be obtained from topographic maps, rainfall reports, and atlases. These regression equations are generally of the following form:

$$RQ_T = aA^bB^cC^d \tag{2-7}$$

where:

$RQ_T$ = T-year rural peak flow

$a$ = regression constant

$b,c,d$ = regression coefficients

$A,B,C$ = basin characteristics

Through a series of studies conducted by the USGS, state highway, and other agencies, rural equations have been developed for all states. The NFF program described in Chapter 12 is a companion software package to implement these equations. These equations should not be used where dams and other hydrologic modifications have a significant effect on peak flows. Many other limitations are presented in USGS documents.

2-3.3.2 **Urban Equations.** Rural peak flow can be converted to urban peak flows with the seven-parameter nationwide urban regression equations developed by the USGS. These equations are shown in Table 2-5. A three-parameter equation has also been developed, but the seven-parameter equation is implemented in NFF. The urban equations are based on urban runoff data from 269 basins in 56 cities and 31 states.

These equations have been thoroughly tested and proven to give reasonable estimates of peak flows having recurrence intervals between 2 and 500 years. Subsequent testing at 78 additional sites in the southeastern United States verified the adequacy of the equations. While these regression equations have been verified, errors may still be approximately 35 to 50 percent when compared to field measurements. More information can be found in the USGS publication, *Flood Characteristics of Urban Watersheds in the United States*.

**Table 2-5. Nationwide Urban Equations Developed by the USGS**

| Equation | Chapter Equation Number |
|---|---|
| $UQ2 = 2.35 A_s^{.41} SL^{.17} (RI2+3)^{2.04} (ST+8)^{-.65} (13-BDF)^{-.32} IA_s^{.15} RQ2^{.47}$ | (2-8) |
| $UQ5 = 2.70 A_s^{.35} SL^{.16} (RI2+3)^{1.86} (ST+8)^{-.59} (13-BDF)^{-.31} IA_s^{.11} RQ5^{.54}$ | (2-9) |
| $UQ10 = 2.99 A_s^{.32} SL^{.15} (RI2+3)^{1.75} (ST+8)^{-.57} (13-BDF)^{-.30} IA_s^{.09} RQ10^{.58}$ | (2-10) |
| $UQ25 = 2.78 A_s^{.31} SL^{.15} (RI2+3)^{1.76} (ST+8)^{-.55} (13-BDF)^{-.29} IA_s^{.07} RQ25^{.60}$ | (2-11) |
| $UQ50 = 2.67 A_s^{.29} SL^{.15} (RI2+3)^{1.74} (ST+8)^{-.53} (13-BDF)^{-.28} IA_s^{.06} RQ50^{.62}$ | (2-12) |
| $UQ100 = 2.50 A_s^{.29} SL^{.15} (RI2+3)^{1.76} (ST+8)^{-.52} (13-BDF)^{-.28} IA_s^{.06} RQ100^{.63}$ | (2-13) |
| $UQ500 = 2.27 A_s^{.29} SL^{.16} (RI2+3)^{1.86} (ST+8)^{-.54} (13-BDF)^{-.27} IA_s^{.05} RQ500^{.63}$ | (2-14) |

where:

| | | |
|---|---|---|
| $UQ_T$ | = | Urban peak discharge for T-year recurrence interval, $ft^3/s$ |
| $A_s$ | = | Contributing drainage area, $mi^2$ |
| $SL$ | = | Main channel slope (measured between points that are 10 and 85 percent of main channel length upstream of site), ft/mi |
| $RI2$ | = | Rainfall intensity for 2-hr, 2-yr recurrence, in/hr |
| $ST$ | = | Basin storage (percentage of basin occupied by lakes, reservoirs, swamps, and wetlands), percent |
| $BDF$ | = | Basin development factor (provides a measure of the hydraulic efficiency of the basin (see description in paragraph 2-3.3.2) |
| $IA$ | = | Percentage of basin occupied by impervious surfaces |
| $RQ_T$ | = | T-year rural peak flow |

The basin development factor (BDF) is a highly significant parameter in the urban equations and provides a measure of the efficiency of the drainage basin and the extent of urbanization. It can be determined from drainage maps and field inspection of the basin. The basin is first divided into upper, middle, and lower thirds. Within each third of the basin, four characteristics must be evaluated and assigned a code of 0 or 1. The four characteristics are: channel improvements; channel lining (prevalence of impervious surface lining); storm drains or storm sewers; and curb and gutter streets.

With the curb and gutter characteristic, at least 50 percent of the partial basin must be urbanized or improved with respect to an individual characteristic to be assigned a code of 1. With four characteristics being evaluated for each third of the basin, complete development would yield a BDF of 12.

Example 2-4

*Given*: The following site characteristics:

- The site is located in Tulsa, Oklahoma.
- The drainage area is 3 square miles ($mi^2$)
- The mean annual precipitation is 38 in.
- Urban parameters (see Table 2-5 for parameter definition):

*SL* = 53 ft/mi

*RI2* = 2.2 in/hr (see National Weather Service Technical Paper 40)

*ST* = 5

*BDF* = 7

*IA* = 35

*Find*: The 2-yr urban peak flow.

*Solution*:

Step 1. Calculate the rural peak flow from the appropriate regional equation.

From Water-Resources Investigations Report 94-4002, the rural regression equation for Tulsa, Oklahoma, is:

$$RQ2 = 0.368A^{.59}P^{1.84} = 0.368(3)^{.59}(38)^{1.84} = 568ft^3/s$$

Step 2. Calculate the urban peak flow using Equation 2-8.

$$UQ2 = 2.35A_s^{.41}SL^{.17}(RI2+3)^{2.04}(ST+8)^{\ .65}(13\quad BDF)^{\ .32}IA_s^{.15}RQ2^{.47}$$

$$UQ2 = 2.35(3)^{.41}(53)^{.17}(2.2+3)^{2.04}(5+8)^{\ .65}(13\quad 7)^{\ .32}(35)^{.15}(568)^{.47} = 747ft^3/s$$

2-3.4 **SCS TR-55 Peak Flow Method.** The SCS (now known as NRCS) peak flow method calculates peak flow as a function of drainage basin area, potential watershed storage, and the time of concentration. An easy to use graphical approach to this method can be found in the TR-55 publication. While some equations are presented in this UFC, graphs, charts, and figures that easily solve the equations are found in TR-55. This rainfall-runoff relationship separates total rainfall into direct runoff, retention, and initial abstraction to yield the following equation for rainfall runoff:

$$Q_D = \frac{(P - 0.2S_R)^2}{P + 0.8S_R} \tag{2-15}$$

where:

$Q_D$ = depth of direct runoff, in.

$P$ = depth of 24-hr precipitation, in. This information is available in most highway agency drainage manuals by multiplying the 24-hr rainfall intensity by 24 hr.

$S_R$ = retention, in.

2-3.4.1 Empirical studies found that $S_R$ is related to soil type, land cover, and the antecedent moisture condition of the basin. These are represented by the runoff curve number, *CN*, which is used to estimate $S_R$ with this equation:

$$S_R = \left[\frac{1000}{CN} - 10\right] \tag{2-16}$$

where:

$CN$ = Curve number, listed in Table 2-6 for different land uses and hydrologic soil types. This table assumes average antecedent moisture conditions. For multiple land use/soil type combinations within a basin, use area weighing (see Example 2-1). Soil maps are generally available through the local jurisdiction or the NRCS. Soils are grouped into categories A through D based on soil characteristics. Soil Group A includes pervious sandy soils, while Soil Group D includes non-pervious rocks and clays. A compete description is provided in TR-55.

2-3.4.2 Peak flow is then estimated with Equation 2-17:

$$q_p = q_u A_k Q_D \tag{2-17}$$

where:

$q_p$ = peak flow, $ft^3/s$

$q_u$ = unit peak flow, ft$^3$/s/mi$^2$/in.

$A_k$ = basin area, mi$^2$

$Q_D$ = runoff depth, in.

The unit peak flow, $q_u$, is calculated with the equations or graphical methods presented in TR-55.

2-3.4.3 The concept of initial abstraction is important to the TR-55 method and can be calculated with the following equation:

$$I_a = 0.2S_R \tag{2-18}$$

$I_a$ = initial abstraction, in.

**Table 2-6. Runoff Curve Numbers for Urban Areas (Average Watershed Condition, $I_a = 0.2S_R$)**

| Land Use Description | | Curve Numbers for Hydrologic Soil Group | | | |
|---|---|---|---|---|---|
| | | A | B | C | D |
| Fully developed urban areas (vegetation established) | | | | | |
| Lawns, open spaces, parks, golf courses, cemeteries, etc. | | | | | |
| Good condition: grass cover on 75 percent or more of the area | | 39 | 61 | 74 | 80 |
| Fair condition: grass cover on 50 to 75 percent of the area | | 49 | 69 | 79 | 84 |
| Poor condition: grass cover on 50 percent or less of the area | | 68 | 79 | 86 | 89 |
| Paved parking lots, roofs, driveways, etc. (excluding right-of-way) | | | | | |
| Streets and roads | | 98 | 98 | 98 | 98 |
| Paved with curbs and storm sewers (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| Gravel (including right-of-way) | | 76 | 85 | 89 | 91 |
| Dirt (including right-of-way) | | 72 | 82 | 87 | 89 |
| Paved with open ditches (including right-of-way) | | 83 | 89 | 92 | 93 |
| | Average % impervious | | | | |
| Commercial and business areas | 85 | 89 | 92 | 94 | 95 |
| Industrial districts | 72 | 81 | 88 | 91 | 93 |
| Row houses, town houses, and residential with lot sizes 0.125 acre or less | 65 | 77 | 85 | 90 | 92 |
| Residential: average lot size | | | | | |
| 0.25 acre | 38 | 61 | 75 | 83 | 87 |
| 0.33 acre | 30 | 57 | 72 | 81 | 86 |
| 0.50 acre | 25 | 54 | 70 | 80 | 85 |

| Land Use Description | | Curve Numbers for Hydrologic Soil Group | | | |
|---|---|---|---|---|---|
| | | A | B | C | D |
| 1 acre | 20 | 51 | 68 | 79 | 84 |
| 2 acres | 12 | 46 | 65 | 77 | 82 |
| Developing urban areas (no vegetation established) | | | | | |
| Newly graded area | | 77 | 86 | 91 | 94 |
| Western desert urban areas: | | | | | |
| Natural desert landscaping (pervious area only) | | 63 | 77 | 85 | 88 |
| Artificial desert landscaping (impervious weed barrier, desert shrub with 1 to 2 in. sand or gravel mulch and basin borders) | | 96 | 96 | 96 | 96 |
| Cultivated agricultural land | | | | | |
| Fallow | | | | | |
| Straight row or bare soil | | 77 | 86 | 91 | 94 |
| Conservation tillage - Poor | | 76 | 85 | 90 | 93 |
| Conservation tillage - Good | | 74 | 83 | 88 | 90 |

2-3.4.4 When ponding or swampy areas occur in a basin, considerable runoff may be retained in temporary storage. The peak flow should be reduced to reflect the storage with Equation 2-19:

$$q_a = q_p F_p \qquad (2\text{-}19)$$

where:

$q_a$ = adjusted peak flow, ft$^3$/s

$F_p$ = adjustment factor, listed in Table 2-7

**Table 2-7. Adjustment Factor ($F_p$) for Pond and Swamp Areas that are Spread Throughout the Watershed**

| Area of Pond or Swamp (percent) | $F_p$ |
|---|---|
| 0.0 | 1.00 |
| 0.2 | 0.97 |
| 1.0 | 0.87 |
| 3.0 | 0.75 |
| 5.0 | 0.72 |

This method has a number of limitations that can have an impact on the accuracy of estimated peak flows:

- The basin should have fairly homogeneous *CN* values.

- The *CN* should be 40 or greater.
- The $t_c$ should be between 0.1 and 10 hr.
- $I_a /P$ should be between 0.1 and 0.5.
- The basin should have one main channel or branches with nearly equal times of concentration.
- Neither channel nor reservoir routing can be incorporated.
- $F_p$ is applied only for ponds and swamps that are not in the $t_c$ flow path.

<u>Example 2-5</u>

*Given*: These physical and hydrologic conditions:

- 1.27 $mi^2$ of fair condition open space and 1.08 $mi^2$ of paved surface (airfield)
- Negligible pond and swamp land
- Hydrologic soil type C
- Average antecedent moisture conditions
- Time of concentration is 0.8 hr.
- 24-hour, Type II rainfall distribution, 10-yr rainfall of 2.8 in.

*Find*: The 10-yr peak flow using the TR-55 peak flow method.

*Solution*:

Step 1 Calculate the composite *CN* using Table 2-6 and Equation 2-2.

$$CN = \sum \frac{(CN_x A_x)}{A} = \frac{[1.27(79) + 1.08(98)]}{(1.27 + 1.08)} = 88$$

Step 2. Calculate the retention, $S_R$, using Equation 2-16.

$$S_R = \left(\frac{1000}{CN} - 10\right) = \left[\left(\frac{1000}{88}\right) - 10\right] = 1.36 \ in.$$

Step 3. Calculate the depth of direct runoff, $Q_D$, using Equation 2-15.

$$Q_D = \frac{(P - 0.2S_R)^2}{(P + 0.8S_R)} = \frac{[2.8 - 0.2(1.36)]^2}{[2.8 + 0.8(1.36)]} = 1.64 \ \ in.$$

$Q_D$ is direct runoff, which means the amount of rainfall available for runoff after losses. Using the direct runoff value and the chart for unit peak discharge found in Chapter 4 of TR-55, the peak discharge can be calculated.

Step 4. Determine $I_a/P$ from $I_a = 0.2S_R$.

$$I_a = 0.2(1.36) = 0.272$$

$$\frac{I_a}{P} = \frac{0.272}{2.8} = 0.097 \ \ say \ \ 0.10$$

Step 5. Calculate peak flow using Equation 2-17.

$$q_p = q_u A_k Q_D \qquad = (410)(2.35)(1.64) = \underline{1580 \ \ ft^3/s}$$

2-4 **DEVELOPMENT OF DESIGN HYDROGRAPHS.** This section discusses methods used to develop a design hydrograph. Hydrograph methods can be computationally involved, so computer programs such as HEC-RAS and HMS (Hydrologic Modeling System), TR-20 (based on SCS Technical Release 20), TR-55, and HYDRAIN are used almost exclusively to generate runoff hydrographs. Hydrographic analysis is performed when flow routing is important, such as in the design of storm water detention, other water quality facilities, and pump stations. Hydrographs can also be used to evaluate flow routing through large storm drainage systems to more precisely reflect flow peaking conditions in each segment of complex systems. See Chapter 12 of this UFC for more information on computer programs for analysis of urban hydrology and hydraulics. HEC-22 contains additional information on hydrographic methods.

2-4.1 **SCS Tabular Hydrograph.** The SCS developed a tabular method that is used to estimate partial composite flood hydrographs at any point in a watershed. This method is generally applicable to small, nonhomogeneous areas that may be beyond the limitations of the Rational Method. It is applicable for estimating the effects of land use change in a portion of the watershed as well as estimating the effects of proposed structures.

2-4.1.1 The SCS tabular hydrograph method is based on a series of unit discharge hydrographs expressed in cubic feet of discharge per second per square mile of watershed per in. of runoff. A series of these unit discharge hydrographs are provided in TR-55 for a range of subarea times of concentration ($T_c$) from 0.1 to 2 hr, and reach travel times ($T_{ti}$) from 0 to 3 hr. One such tabulation is provided in Table 2-8.

## Table 2-8. Tabular Hydrograph Unit Discharges for Type II Rainfall Distributions (English Units)

**Tabular hydrograph unit discharges (csm/in) for type II rainfall distributions (Taken from SCS TR-55 Manual)**

| TRVL TIME (HR) | HYDROGRAPH TIME (HOURS) 11.0 | 11.3 | 11.6 | 11.9 | 12.0 | 12.1 | 12.2 | 12.3 | 12.4 | 12.5 | 12.6 | 12.7 | 12.8 | 13.0 | 13.2 | 13.4 | 13.6 | 13.8 | 14.0 | 14.3 | 14.6 | 15.0 | 15.5 | 16.0 | 16.5 | 17.0 | 17.5 | 18.0 | 19.0 | 20.0 | 22.0 | 26.0 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| IA/P = 0.10 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | IA/P = 0.10 | | | | |
| 0.0 | 17 | 23 | 32 | 57 | 94 | 170 | 308 | 467 | 529 | 507 | 402 | 297 | 226 | 140 | 96 | 74 | 61 | 53 | 47 | 41 | 36 | 32 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .10 | 16 | 22 | 30 | 51 | 80 | 140 | 252 | 395 | 484 | 499 | 434 | 343 | 265 | 162 | 108 | 80 | 65 | 55 | 49 | 42 | 36 | 33 | 29 | 26 | 23 | 21 | 20 | 19 | 16 | 14 | 12 | 0 |
| .20 | 14 | 19 | 25 | 38 | 47 | 69 | 116 | 207 | 332 | 434 | 477 | 449 | 378 | 238 | 149 | 101 | 77 | 62 | 53 | 45 | 39 | 34 | 30 | 27 | 24 | 22 | 20 | 19 | 17 | 14 | 12 | 0 |
| .30 | 13 | 18 | 24 | 35 | 43 | 60 | 97 | 170 | 278 | 382 | 446 | 448 | 401 | 270 | 171 | 114 | 83 | 66 | 56 | 46 | 40 | 34 | 31 | 27 | 24 | 22 | 20 | 19 | 17 | 15 | 12 | 0 |
| .40 | 12 | 15 | 21 | 29 | 33 | 40 | 53 | 83 | 141 | 233 | 332 | 408 | 434 | 361 | 243 | 157 | 107 | 79 | 64 | 51 | 43 | 36 | 32 | 28 | 25 | 22 | 21 | 20 | 17 | 15 | 12 | 0 |
| .50 | 11 | 15 | 20 | 28 | 31 | 37 | 48 | 71 | 118 | 194 | 286 | 367 | 412 | 378 | 271 | 178 | 119 | 86 | 68 | 53 | 44 | 37 | 32 | 29 | 25 | 23 | 21 | 20 | 17 | 15 | 12 | 0 |
| .75 | 9 | 11 | 14 | 19 | 21 | 24 | 27 | 31 | 37 | 49 | 74 | 118 | 182 | 319 | 374 | 328 | 244 | 169 | 117 | 76 | 56 | 43 | 35 | 31 | 28 | 25 | 22 | 21 | 18 | 16 | 12 | 1 |
| 1.0 | 7 | 9 | 12 | 16 | 17 | 19 | 21 | 24 | 27 | 32 | 40 | 55 | 83 | 188 | 309 | 359 | 322 | 245 | 172 | 102 | 68 | 49 | 38 | 32 | 29 | 26 | 23 | 21 | 19 | 16 | 12 | 1 |
| 1.5 | 5 | 7 | 8 | 11 | 12 | 13 | 14 | 15 | 17 | 19 | 21 | 23 | 27 | 43 | 89 | 175 | 269 | 322 | 309 | 225 | 140 | 77 | 49 | 38 | 32 | 29 | 25 | 23 | 20 | 17 | 13 | 5 |
| 2.0 | 3 | 4 | 6 | 7 | 8 | 8 | 9 | 10 | 10 | 11 | 12 | 14 | 15 | 18 | 23 | 35 | 65 | 123 | 202 | 297 | 280 | 181 | 88 | 52 | 39 | 33 | 29 | 26 | 21 | 19 | 14 | 10 |
| 2.5 | 2 | 3 | 4 | 5 | 5 | 6 | 6 | 7 | 7 | 8 | 9 | 9 | 10 | 12 | 15 | 18 | 24 | 36 | 66 | 150 | 244 | 278 | 171 | 87 | 52 | 39 | 33 | 29 | 23 | 20 | 15 | 11 |
| 3.0 | 1 | 1 | 2 | 3 | 3 | 4 | 4 | 4 | 5 | 5 | 6 | 6 | 7 | 8 | 9 | 11 | 13 | 16 | 20 | 37 | 86 | 198 | 263 | 182 | 96 | 56 | 40 | 33 | 26 | 21 | 16 | 11 |
| IA/P = 0.30 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | IA/P = 0.30 | | | | |
| 0.0 | 0 | 0 | 0 | 1 | 9 | 53 | 157 | 314 | 433 | 439 | 379 | 299 | 237 | 159 | 118 | 95 | 81 | 71 | 65 | 56 | 50 | 46 | 42 | 38 | 34 | 31 | 30 | 28 | 25 | 22 | 19 | 0 |
| .10 | 0 | 0 | 0 | 0 | 1 | 6 | 37 | 117 | 248 | 372 | 416 | 391 | 330 | 218 | 150 | 113 | 92 | 79 | 70 | 60 | 53 | 47 | 43 | 39 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .20 | 0 | 0 | 0 | 0 | 1 | 4 | 26 | 87 | 194 | 313 | 382 | 388 | 349 | 244 | 167 | 122 | 97 | 82 | 72 | 62 | 54 | 48 | 43 | 39 | 35 | 32 | 30 | 29 | 26 | 22 | 19 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 19 | 64 | 151 | 259 | 341 | 372 | 316 | 223 | 156 | 117 | 94 | 80 | 67 | 58 | 50 | 45 | 41 | 36 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 13 | 47 | 116 | 211 | 298 | 354 | 328 | 245 | 172 | 127 | 100 | 83 | 69 | 59 | 51 | 45 | 41 | 37 | 33 | 31 | 29 | 26 | 23 | 19 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 9 | 34 | 89 | 170 | 255 | 341 | 303 | 225 | 161 | 120 | 96 | 76 | 64 | 54 | 47 | 42 | 38 | 34 | 31 | 30 | 27 | 24 | 19 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 14 | 41 | 89 | 152 | 270 | 305 | 268 | 207 | 155 | 118 | 87 | 70 | 57 | 48 | 44 | 39 | 35 | 32 | 30 | 27 | 24 | 19 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 7 | 22 | 98 | 212 | 295 | 285 | 237 | 181 | 120 | 88 | 67 | 53 | 46 | 42 | 38 | 34 | 31 | 28 | 25 | 19 | 2 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 30 | 95 | 183 | 249 | 265 | 217 | 152 | 96 | 66 | 53 | 46 | 41 | 37 | 34 | 30 | 26 | 20 | 8 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 18 | 59 | 125 | 221 | 245 | 182 | 105 | 69 | 54 | 47 | 42 | 38 | 32 | 28 | 22 | 16 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 5 | 21 | 84 | 174 | 230 | 172 | 103 | 69 | 54 | 46 | 42 | 34 | 30 | 23 | 18 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 13 | 56 | 157 | 217 | 163 | 101 | 68 | 53 | 46 | 37 | 31 | 25 | 18 |
| IA/P = 0.50 | | | | | | | | | | | | | * * * TC =0.5 HR * * * | | | | | | | | | | | | | | | IA/P = 0.50 | | | | |
| 0.0 | 0 | 0 | 0 | 0 | 0 | 2 | 26 | 89 | 170 | 217 | 229 | 200 | 179 | 144 | 119 | 104 | 93 | 85 | 78 | 70 | 64 | 59 | 55 | 51 | 46 | 43 | 41 | 40 | 36 | 32 | 28 | 0 |
| .10 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 18 | 65 | 135 | 190 | 216 | 205 | 170 | 137 | 115 | 101 | 91 | 83 | 74 | 67 | 61 | 56 | 52 | 47 | 44 | 42 | 40 | 36 | 32 | 28 | 0 |
| .20 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 12 | 47 | 106 | 162 | 198 | 203 | 178 | 145 | 121 | 105 | 94 | 85 | 76 | 68 | 61 | 57 | 52 | 48 | 44 | 42 | 40 | 37 | 32 | 28 | 0 |
| .30 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 8 | 34 | 82 | 135 | 177 | 194 | 168 | 139 | 117 | 102 | 92 | 80 | 71 | 63 | 58 | 54 | 49 | 45 | 43 | 41 | 37 | 33 | 28 | 0 |
| .40 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 6 | 25 | 63 | 111 | 155 | 189 | 174 | 146 | 122 | 106 | 94 | 82 | 73 | 64 | 58 | 54 | 50 | 45 | 43 | 41 | 37 | 33 | 28 | 0 |
| .50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 18 | 48 | 90 | 133 | 184 | 177 | 152 | 128 | 110 | 97 | 84 | 74 | 65 | 59 | 55 | 50 | 45 | 43 | 41 | 38 | 33 | 28 | 0 |
| .75 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 22 | 47 | 180 | 142 | 169 | 164 | 144 | 124 | 108 | 91 | 79 | 68 | 61 | 56 | 51 | 47 | 44 | 42 | 38 | 34 | 28 | 0 |
| 1.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 11 | 51 | 112 | 155 | 166 | 154 | 134 | 109 | 91 | 76 | 65 | 59 | 54 | 49 | 45 | 43 | 39 | 35 | 28 | 2 |
| 1.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 16 | 50 | 97 | 136 | 154 | 145 | 121 | 95 | 75 | 64 | 58 | 54 | 49 | 45 | 41 | 37 | 29 | 10 |
| 2.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 18 | 47 | 86 | 134 | 146 | 125 | 94 | 75 | 64 | 58 | 53 | 49 | 42 | 39 | 31 | 21 |
| 2.5 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 11 | 44 | 95 | 140 | 127 | 97 | 77 | 65 | 58 | 54 | 45 | 41 | 33 | 26 |
| 3.0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 7 | 29 | 86 | 135 | 122 | 95 | 76 | 65 | 58 | 49 | 43 | 35 | 27 |
| RAINFALL TYPE = II | | | | | | | | | | | | | * * * TC = 0.5 HR * * * | | | | | | | | | | | | | | | | | | | |

Copied from TR-55[13]

2-4.1.2 The hydrograph ordinates for a specific time are determined by multiplying the runoff depth, the subarea, and the tabular hydrograph unit discharge value for that time as determined from the tables. See Equation 2-20:

$$q = q_t A Q_D \tag{2-20}$$

where:

$q$ = hydrograph ordinate for a specific time, $ft^3/s$

$q_t$ = tabular hydrograph unit discharge from appropriate table, $ft^3/s/mi^2/in$

$A$ = sub-basin drainage area, $mi^2$

$Q_D$ = runoff depth, in.

2-4.1.3 The TR-55 publication provides a detailed description of the tabular hydrograph method. In developing the tabular hydrograph, the watershed is divided into homogeneous subareas. Input parameters required for the procedure include: (1) the 24-hr rainfall amount, in., (2) an appropriate rainfall distribution (I, IA, II, or III), (3) the runoff curve number, CN, (4) the time of concentration, $T_c$, (5) the travel time, $T_{ti}$, and (6) the drainage area, $mi^2$, for each subarea. The 24-hr rainfall amount, rainfall distribution, and the runoff curve number are used in Equations 2-15 and 2-16 to determine the runoff depth in each subarea. The product of the runoff depth times drainage is multiplied times each tabular hydrograph value to determine the final hydrograph ordinate for a particular subarea. Subarea hydrographs are then added to determine the final hydrograph at a particular point in the watershed. Example 2-6 provides an illustration of the use of the tabular hydrograph method.

2-4.1.4 These assumptions and limitations are inherent in the tabular method:

- The total area should be less than 2000 acres. Typically, subareas are far smaller than this because the subareas should have fairly homogeneous land use.
- The travel time, $T_{ti}$, is less than or equal to 3 hr.
- The time of concentration, $t_c$, for any given subarea is less than or equal to 2 hr.
- The drainage areas of individual subareas differ by less than a factor of 5.

Example 2-6

*Given*: A watershed with three subareas. Subareas 1 and 2 both drain into Subarea 3. Consider the basin data for the three subareas:

| Subarea | Area ($mi^2$) | $t_c$ (hr) | $T_{ti}$ (hr) | *CN* |
|---|---|---|---|---|
| 1 | 0.386 | 0.5 | --- | 75 |
| 2 | 0.193 | 0.5 | --- | 65 |
| 3 | 0.927 | 0.5 | 0.20 | 70 |

A time of concentration, $t_c$, of 0.5 hr, an $I_a/P$ value of 0.10, and a Type II storm distribution are assumed for convenience in all three subareas. The travel time applies to the reach for the corresponding area; therefore, the travel time, $T_{ti}$, in Subarea 3 will apply to the tabular hydrographs routed from Subareas 1 and 2.

*Find*: The outlet hydrograph for a 5.9-in. storm.

*Solution*:

Step 1. Calculate the retention for each of the subareas using Equation 2-16.

$$S_R = \left(\frac{1000}{CN} - 10\right)$$

Subarea 1. $$S_R = \left(\frac{1000}{75} - 10\right) = 3.33 \text{ in.}$$

Subarea 2. $$S_R = \left(\frac{1000}{65} - 10\right) = 5.38 \text{ in.}$$

Subarea 3. $$S_R = \left(\frac{1000}{70} - 10\right) = 4.29 \text{ in.}$$

Step 2. Calculate the depth of runoff for each of the subareas using Equation 2-15.

$$Q_D = \frac{(P - 0.2S_R)^2}{P + 0.8S_R}$$

Subarea 1. $$Q_D = \frac{[5.9 - 0.2(85)]^2}{[5.9 + 0.8(85)]} = 3.2 \text{ in.}$$

Subarea 2. $$Q_D = \frac{[5.9 - 0.2(137)]^2}{[5.9 + 0.8(137)]} = 2.28 \text{ in.}$$

Subarea 3. $$Q_D = \frac{[5.9 - 0.2(109)]^2}{[5.9 + 0.8(109)]} = 2.72 \text{ in.}$$

Step 3. Calculate ordinate values using Equation 2-20: $q = q_t A Q_D$.

Multiply the appropriate tabular hydrograph values ($q_t$) from Table 2-8 by the subarea areas ($A$) and runoff depths ($Q$) and sum the values for each time to give the composite hydrograph at the end of Subarea 3. For example, the hydrograph flow contributed from Subarea 1 ($t_c$ = 0.5 hr, $T_{ti}$ = 0.20 hr) at 12.0 hr is calculated as the product of the tabular value, the area, and the runoff depth, or 47 (0.386)3.2 = 58 $ft^3/s$.

Table 2-9 lists the subarea and composite hydrographs. Please note that this example does not use every hydrograph time ordinate.

**Table 2-9. Subarea and Composite Hydrographs**

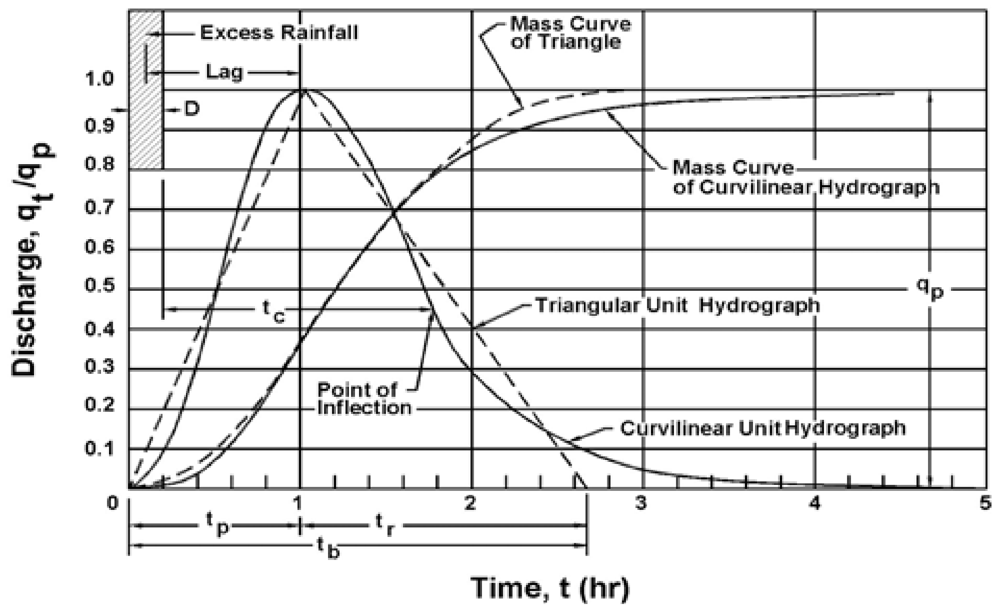
| | Flow at Specified Time ($ft^3/s$) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Subarea | **11 (hr)** | **12 (hr)** | **12.2 (hr)** | **12.4 (hr)** | **12.5 (hr)** | **12.6 (hr)** | **12.8 (hr)** | **13 (hr)** | **14 (hr)** | **16 (hr)** | **20 (hr)** |
| 1 | 17 | 58 | 143 | 410 | 536 | 584 | 466 | 294 | 65 | 33 | 17 |
| 2 | 6 | 21 | 51 | 146 | 191 | 210 | 166 | 105 | 23 | 12 | 6 |
| 3 | 43 | 238 | 778 | 1337 | 1281 | 1016 | 571 | 354 | 119 | 66 | 35 |
| Total | 66 | 317 | 972 | 1893 | 2008 | 1815 | 1203 | 753 | 207 | 111 | 58 |

2-4.2 **SCS Synthetic Unit Hydrograph (UH)**. The SCS developed a synthetic UH procedure that has been widely used in conservation and flood control work. The UH used by this method is based upon an analysis of a large number of natural UHs from a broad cross section of geographic locations and hydrologic regions.

2-4.2.1 This method is easy to apply. The only parameters that need to be determined are the peak discharge and the time to peak ($t_p$). A standard UH is constructed using these two parameters.

2-4.2.2 For the development of the SCS UH, the curvilinear UH is approximated by a triangular UH that has similar characteristics. Figure 2-4 shows a comparison of the two dimensionless UHs. Even though the time base ($t_b$) of the triangular UH is 8/3 of the time to peak, $t_p$, and the $t_b$ of the curvilinear UH is five times the $t_p$, the area under the two UH types is the same.

**Figure 2-4. Dimensionless Curvilinear SCS Synthetic Unit Hydrograph and Equivalent Triangular Hydrograph**




2-4.2.3 The area under a hydrograph equals the volume of direct runoff, $Q_D$, which is 1 inch for a UH. The peak flow is calculated using Equation 2-21:

$$q_p = \frac{K_c A_k Q_D}{t_p} \quad (2\text{-}21)$$

where:

$q_p$ = peak flow, $ft^3/s$

$A_k$ = drainage area, $mi^2$

$Q_D$ = volume of direct runoff ( = 1 for unit hydrograph), in.

$t_p$ = time to peak, hr

$K_c$ = 483.5

2-4.2.4 The constant 483.5 reflects a UH that has 3/8 of its area under the rising limb. For mountainous watersheds, the fraction could be expected to be greater than 3/8, and therefore the constant may be near 603.5. For flat, swampy areas, the constant may be on the order of 301.7.

Time to peak, $t_p$, can be expressed in terms of time of concentration, $t_c$, as in Equation 2-22:

$$t_p = \frac{2}{3} t_c \tag{2-22}$$

Expressing $q_p$ in terms of $t_c$ rather than $t_p$ yields:

$$q_p = \frac{K_c A_k Q_D}{t_c} \tag{2-23}$$

where $K_c$ = 725.25

<u>Example 2-7</u>

*Given*: The following watershed conditions:

- The watershed is commercially developed.
- Watershed area = 0.463 mi$^2$
- Time of concentration, $t_c$, = 1.34 hr
- $Q_D$ = 1.0 in.

*Find*: The triangular SCS UH.

*Solution*:

Step 1. Calculate peak flow using Equation 2-23.

$$q_p = \frac{K_c A_k Q_D}{t_c}$$

$$= \frac{725.25\ (0.463)\ (1.0)}{1.34}$$

$$= 250.59 \text{ ft}^3/\text{s}$$

Step 2. Calculate the time to peak, $t_p$, using Equation 2-22.

$$t_p = \frac{2}{3} t_c = \frac{2}{3}(1.34) = 0.893 \text{ hr}$$

Step 3. Calculate the time base, $t_b$, of the UH.

Step 4. Draw the resulting triangular UH (see Figure 2-5).

$$t_b = \frac{8}{3}(0.893) = 2.38 \text{ hr}$$

***NOTE:*** The curvilinear SCS UH is more commonly used and is incorporated into many computer programs.

**Figure 2-5. Example: The Triangular Unit Hydrograph**